BEHAVIOUR STUDY OF R.C.C. BEAMS WITH HIGH YIELD STRENGTH DEFORMED BARS UNDER PULSATING LOADS

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By
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CERTIFICATE

This is to certify that the thesis entitled 'Behaviour Study of R.C.C. Beams With High Yield Strength Deformed Bars Under Pulsating Loads' by Sitangshu Mukhopadhyay is a record of work carried out under my supervision and has not been submitted elsewhere for a degree.

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LIST OF SYMBOLS AND ABBREVIATIONS

HSD bar	=	High yield strength deformed bar
CPM	=	Cycles per minute
σ_{cu}	=	28 days cube strength of concrete
σ_{yp}	=	Yield on proof stress
σ_{ult}	=	Ultimate stress
σ^{E}_{yp}	=	Experimental yield or proof stress
σ^{E}_{ult}	=	Experimental ultimate stress
P	=	Load
P_U	=	Ultimate load
P^T_U	=	Theoretical ultimate load
P^E_U	=	Experimental ultimate load
P_1	=	Lower limit of variable pulsating load
P_2	=	Upper limit of variable pulsating load
P_{peak}	=	Second upper limit; peak load level of variable pulsating load
σ_1	=	Stress level corresponding to lower load P_1
σ_2	=	Stress level corresponding to upper load P_2
σ_{peak}	=	Overstress or peak stress level corresponding to peak load (P_{peak})

C	=	Compressive force on the hinge
T	=	Tension developed in the HSD bar
T_{ult}	=	Ultimate tensile force
d	=	effective depth
s	=	Centre to centre distance between roller and bar.
A_{st}	=	Area of tensile reinforcement
C.L.F	=	Combined load factor
M	=	Bending moment
M_U	=	Ultimate moment

ABSTRACT

It may be economical to design a structure treating peak loads (wind or earthquake load or similar situations) as overloads. Use of high yield strength deformed bars (HSD bars) in R.C. Structures minimizes the area of reinforcements. Welding in reinforcements is permitted to save bond or lap length by different codes of practice. An attempt is made towards the study of the effect of variable loads (D.L., L.L. and W.L. or E.L. or overload) on R.C.C. Beams with HSD bars with or without welding. Lap and Butt joints of 25 Ø HSD bars are chosen as reinforcement. The effect of variable pulsating loads on deformations, crack propagation and the ultimate strength of R.C.C. beams with 25 Ø HSD bars (with or without weld) is studied.

Results of the experimental study have shown that butt welded joint subjected to stress levels 1163, 2292 and 3060 kg/cm^2 (i.e; 20%, 40% and 53% of experimental ultimate stress) sustained 2.43 million cycles and cumulative damage was very small. 25 Ø HSD bar without weld subjected to stress levels 20%, 62% and 75% of experimental ultimate stress (σ^E_{ult} = 5778 kg/cm^2), failed after 0.68 million

cycles. R.C. Beam with unwelded HSD bar subjected to load levels 16.5%, 60% and 74% of ultimate load P_U sustained 1 million cycles and the cumulative damage after 1 million cycle was 1/750th of span length. Load factor 1.5 for D.L. and L.L., and load factor 1.2 for combination of D.L., L.L. and W.L. or E.L.were chosen. Beams with butt welded HSD bars did not stand against the same load levels. The section of the bar next to the weld failed under pulsating loads of 0.2 to 0.5 million cycles.

CHAPTER I

GENERAL INTRODUCTION

1.1 INTRODUCTION :

In general, Loads acting on structures could broadly be classified as : (1) deterministic loads and (2) probabilistic loads. Self-weights are usually deterministic because they are fixed in position and magnitude. For all practical purposes without any loss of accuracy or economy some superimposed load fixed in position and magnitude can be treated as deterministic. Environmental loads like Wind and Earthquake loads are probabilistic in nature and designer has very little control on them. Though the actual occurance of such environmental loads are random, sometimes peak magnitudes of such loads can be predicted to a certain accuracy. On some structures like bridges, the occurance of live loads is again random phenomenon. Most of the live loads which act on different type of structures are probabilistic in nature rather than deterministic. The probabilistic loads coming on a particular structure can be idealized as deterministic load whereby the frequency of occurance of such load looses its significance even though the magnitude may be obtained

by a statistical basis either as peak or mean normal load. In a limited extent, the probability of occurance of such probabilistic load is reflected by the higher permissible stresses under combined load conditions of working stress design and similerly lower load factors under combined load conditions of ultimate strength design.

Adequate safety and serviceability of a structure are the primary aim of a structural designer. Since present trends in design are more and more towards optimization of materials and overall cost of the structures, an adequate safety with maximum economy should be the aim of a designer. Minimum load factors or maximum permissible stresses that ensure safety and serviceability of a structure will yield economy in structural design.

Previously ordinary mild steel reinforcements were used in Reinforced Concrete structures. Later on use of high strength deformed bars has come up because of their high strength. These days high strength deformed bars have practically replaced smooth mild steel bars in several countries. It has been seen in different cases that the use of high strength deformed bars gives economy in Reinforced Concrete structures though the cost of manufacture of high yield strength deformed bars with

yield stress 4250 kg/cm^2 is only nominally higher than the ordinary mild steel bars with yield stress 2600 kg/cm^2.

Some of the advantages of using high strength deformed bars are given below :-

(i) The effectiveness of reinforcement in a concrete member is roughly proportional to the product of yield stress and cross sectional area of the reinforcement. Since yield stress is higher in case of high strength deformed bars, substantial saving in the amount of steel is achieved resulting total economy.

(ii) Reduction of lap lengths and end hooks, resulting from the increased bond strength.

(iii) Reduced crack width in flexural members, resulting from large number of fine cracks more evenly distributed in the members.

(iv) Ease of placing concrete due to the reduction in total number of bars to be used in a section and elimination of end hooks.

(v) Steel reduction sometimes permit reduction in width of major girders which in turn reduces dead loads.

(vi) Reduction in dead weights, shuttering costs as smaller cross-section is involved etc.

High strength deformed bars are produced by either

(a) Cold working of mild steel deformed bars or

(b) hot rolling of steel whose mechanical properties have been improved by the addition of suitable alloying elements.

Cold twisted deformed bars have the following properties :-

Approximate chemical composition :

High strength deformed bar is made from basically soft mild steel (I.S. 226) with maximum limit as -

Carbon	Sulphur	Phosphorous
0.25%	0.055%	0.055%

Physical Properties (I.S. 1786) :

Size	Proof Stress	Ultimate Tensile stress	Elongation on gauge length $5.65\sqrt{Area}$
All	4250 kg/cm^2	4950 kg/cm^2	14.5%

Permissible Stress (I.S. 456 - 1968)

Tension	Compression	Shear	Bond
For dia. $\leqslant$ 20 mm 2300 kg/cm^2 For dia. $>$ 20 mm 2100 kg/cm^2	1750 kg/cm^2	1750 kg/cm^2	40% more than round bars.

High strength deformed bars also called cold twisted deformed bars, have no definite yield point. Stress strain behaviour of HSD bars indicates its tendency towards brittleness. IS-456-1964 (second revision) Cl. 4.5.2 permits welded joint in reinforcements. Welding of reinforcement should be done in accordance with the recommendations of IS : 2751 - 1966 with special precautions in the welding of cold worked reinforcing bars. In case of HSD bars, the bond is 40% more than round bars and hence smaller bond length or lap length is needed. For smaller diameter of bar to provide bond length or lap length is no problem. But for higher diameter of bars bond or lap length is higher and hence welding of reinforcement may result saving of material and ease of placing concrete.

1.2 DESIGN CRITERIA :

Any structural design consists of the determination of its general shape and all details of a particular structure, such that it will perform the required function and should safely withstand all the influences coming on it throughout its useful life. Different types of loads and other environmental agents such as temperature, weather, settlement of foundations and other effects are considered as the influences.

The main items of practical interest in any structural design are :

(a) the strength of the structure

(b) the deformation, such as deflections and extent of cracking when loaded under service conditions.

Generally there are three different approaches of structural design :

(i) Working Stress Design : It is a design criterion which directs the attention to a sets of permissible stresses within the structural members under working loads. In this method, permissible stresses are established as some fraction of the yield stress or proof stress of

the materials. Under working load the stresses in the member are not allowed to exceed these permissible values. The margin of safety in this method is provided in the allowable stresses. The working loads are generally due to self weight and the live loads on a structure. Sometimes the structure is overloaded due to occurrence of various other types of loads such as wind loads, snow loads, earthquake loads etc. When the combined effect of wind and earthquake load etc. is included, a higher stress value above the permissible stress level is permitted. This increase in permissible stress accounts for the less probability of the occurance of the above mentioned combination of loading. The allowable stresses and the higher permissible values beyond these allowable stresses for concrete when combined loads act, as per different codes of practice, are given in Table 1.1 and 1.2.

(ii) Ultimate Strength Design : It is a design method which is based on strength capacity of the member just before failure condition. In this method, structural members are designed so that the full strength of the cross-section is just utilised when the ultimate load is applied, which is obtained by multiplying the working load by a load factor greater than unity. In this method,

TABLE 1.1 PERMISSIBLE STRESSES IN CONCRETE IN BENDING

Codes	Permissible Stresses
A.C.I (1)*	0.45 σ_{cy} where σ_{cy}. is cylinder strength in 28 days.
I.S. (2)	Approximately 0.33 σ_{cu}
B.S. (11)	0.33 σ_{cu} where σ_{cu} is cube strength in 28 days.

TABLE 1.2 HIGHER PERMISSIBLE STRESSES IN CONCRETE

Codes	Higher Permissible Stresses
A.C.I. (1), I.S. (2)	$33\frac{1}{3}$ percent more than the permissible stress value.
B.S. (11)	25 percent more than the permissible stress value

* The numerals in the brackets indicate the reference number.

margin of safety is introduced by load factors.

(iii) Limit State Design : Limit State Design method evaluates the load carrying capacity of the structure with respect to some specified limit states. In this method safety and serviceability of any structural member is ensured by taking a permissible value of the stress acting on it in service conditions as less than the ultimate stress.

A limit state defines the state of a member, in which it looses its ability to withstand external loads, or suffers excessive deformation or local damage. All relevant limit states should be considered in the design so as to ensure an adequate degree of safety and serviceability. The usual procedure adopted in this approach is to design the structure for a critical limit state and then to check for the remaining limit states.

The magnitude of the permissible stress is determined according to the variation of several factors; such as the load, the mechanical properties of the materials, and the service conditions of the member. Two partial safety factors are used, one for material strength and other for loading. These safety factors have different values

according to material, type of loading and the limit state being considered.

The three limit states normally considered in the design are :

(a) Collapse :

Collapse of the structure, or part of the structure, which arises from rupture of one or more critical sections, from over-turning or from buckling by elastic or plastic instability including the effect of sway where appropriate, should be considered.

(b) Deflection :

The deflection of the structure or any part of the structure which impair the appearance or efficiency of the structure, affect adversely nonload bearing members or finishes or cause discomfort or alarm to the users of the structure.

(c) Cracks and Local Damage :

Local damage, such as cracking or splitting of concrete which impairs the strength, efficiency and appearance of the structure through reduction in stiffness, corrosion of reinforcement etc.

1.3 LOAD FACTORS :

In the ultimate strength design method and limit design methods load factors are used to provide margin of safety depending upon the purpose of the structure. Load factors are selected with respect to the probability of increase in the loads within the lifetime of a structure.

Load factors are selected to take the account of the following :

(i) The variation in dimensional accuracy achieved in construction.

(ii) The variable workmanship in construction.

(iii) Possibility of having the strength of material different from that specified by designer.

(iv) Possible unusual increase in load beyond those considered in deriving the characteristic load.

(v) Inaccuracies in assessment of effects of loading and unforeseen stress redistributions within the structure etc.

The load factors used are different for different types of loads, which is supposed to act on the structure in its life time. In general, these are dead loads, live

loads, wind loads, earthquake loads etc. Load factors specified by different codes of practice for different loads are given in Table 1.3. In limit design ,IS 456 draft code 1978 specifies two sets of partial safety factors based upon collapse and serviceability and are given in Table 1.4.

1.4 SAFETY AND SERVICEABILITY :

To serve the purpose, a structure should be safe against collapse and serviceable while in use. Serviceability of a structure requires that deflections and cracks should be small and within some tolerable limits. Safety requires that the strength of the structure should be adequate to withstand all the loads coming on it. The working stress provisions introduce the safety factor by means of allowable stresses while the ultimate strength provision or limit state provision introduce safety factor by means of load factors and capacity reduction factors. This capacity reduction factor is different for different materials and the theoretical strength of a structural member for perfect material and workmanship is predicted by considering capacity reduction factors for materials and load factors for the loads.

TABLE 1.3 LOAD FACTORS FOR DIFFERENT COMBINATION OF LOADS

Sl. No.	Specification	Load Factors					Material Reduction Coefficient
		Live Load		Combined Load			
		N_d	N_l	N_d	N_l	N_w	
1.	I.S. (2)	1.5	2.2	1.5	2.2	0.5	
				or,			1
				1.5	0.5	2.2	
2.	A.C.I (1)	1.5	1.8	1.5	1.25	1.25	
3.	B.S. (11)	1.4	1.6	1.25	1.25	1.2	

N = Load factors, subscripts d, l, w indicate dead, live and wind or earthquake loads respectively.

TABLE 1.4 LOAD FACTORS AS PER IS-456- REVISED DRAFT 1978

Load Combination	Limit state of collapse			Limit states of serviceability		
	D.L.	L.L.	W.L.	D.L.	L.L.	W.L.
D.L.+ L.L.	1.5	1.5	-	1.0	1.0	-
D.L.+ W.L.	1.5 or 0.9*	-	1.5	1.0	-	1.0
D.L. + L.L. + W.L.	1.2	1.2	1.2	1.0	0.8	0.8

* This value is to be considered when stability against overturning or stress reversal is critical.

1.5 LIVE LOADS :

Self weight of the structure and any other type of load which is fixed in position and magnitude is called dead load. Any load whose application and release occurs from time to time is in general called Live load. Since Live load is a time dependent phenomenon and random in nature, exact probability of occurence and magnitude cannot be predicted. Live loads may be applied in a repeated cyclic manner causing elastic or plastic strains of alternative sense or of same sense developed in a structure.

1.6 OVERLOAD CONDITIONS :

Live loads acting on the structure are usually considered deterministic in design though the actual occurance and magnitude of such loads are not truly known to the designer. The effects of Wind and Earthquake forces on the structures are random and unknown to the designer. These loads may have some type of probable cycle of occurence. The velocity of the wind reaches different peak value in different time intervals. For examplo, the maximum expected wind velocity at a particular area from previous observations for a

certain period of time, may reach higher velocities in larger interval of time. Since velocity of wind is proportional to the force exerted by wind on a structure, reaching higher velocity means higher load compared to the expected maximum load, and these higher loads can be termed as overloads. Similar things may also happen due to earthquakes.

When the stress level at any point of the structure increases beyond the normal permissible stress level due to loads other than normal working loads, the structure is said to be over-loaded. The traffic on a bridge varies from time to time. Traffic loads are not applied in a regular cyclic pattern. They occur intermittently with rest periods between individual loads as pulses over the whole life of the structure. Let us take an example of high-way bridge subjected to normal traffic. The rate of loading varies with the speed of the traffic and number of vehicles. They are subjected to daily and seasonal variations. In addition to these, there are other possibilities of a bridge structures being overloaded. (1) Vehicles carrying heavy ammunition (the weight of which are above the live load for which the bridge is designed) across the bridge at the time of war.

(2) Violation of the loading limits, fixed by local bye-laws or by the codes, by the truck drivers etc.

(3) Transportation of heavy equipment for heavy industry.

To design a structure to the exact load pattern is not possible in a deterministic design. Hence a proper idealization and a selection of design load has to be made. A typical load characteristic is shown in Fig. 1.1.

A particular load frequency may be idealized either by energy criteria or by static criteria. An approximate typical idealized load frequency is shown in Fig. 1.2. It is clear from the figure that different peak loads occur in different time intervals during the life time of a structure and from the study of the frequency of different peak loads which are likely to occur in the life time of a structure, a normal design load is to be selected such that it will neither fail in its life time due to unexpected overloads nor be over designed. For example referring to Fig. 1.2 if level 'A' is taken as the normal design load, then the structure may be called overdesigned because the occurence of such load may be one in million. In that case, the structure is very much under stressed for most part of its life. A normal design load may be defined as the

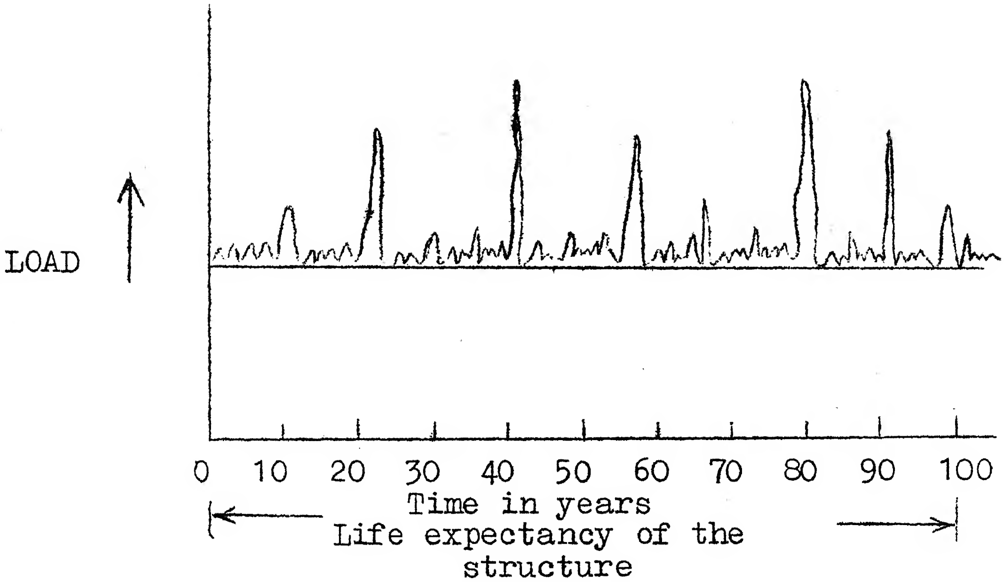


Fig. 1.1 Typical Load Characteristics

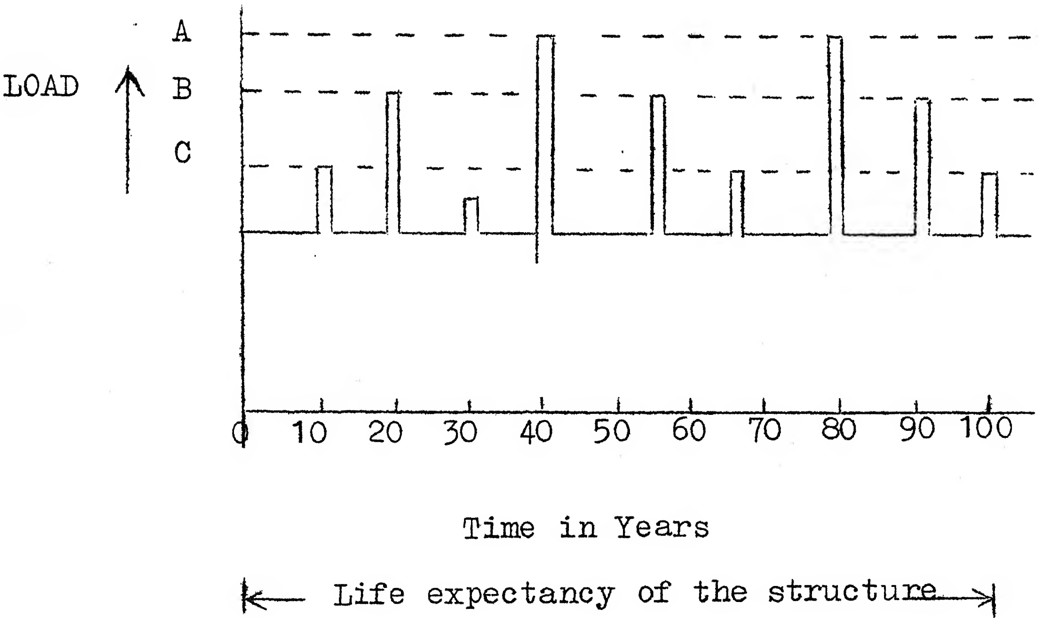


Fig. 1.2 Idealised Load Characteristics

load at which full stresses are permitted in the working stress design or full load factors are applied in the ultimate load design. When load level 'C' is chosen as the normal design load, the structure may suffer from serious damage or may collapse when the peak load level 'A' comes on the structure during its life time. Hence a design load level 'B' may be selected such that it will be able to withstand the heavy stresses caused by the load level 'A' in its life period and at the same time will not be overdesigned. The load corresponding to load level 'A' may be called as overload since the permissible stresses are exceeded in case load level 'B' is selected as design load. The choice of any design load influences the economy of the structure and hence a careful consideration must be given to its selection.

1.7 LITERATURE REVIEW :

Tests have been conducted on Reinforced concrete beams under repeated load. Rasmussen (10) carried out tests of R.C. beams reinforced with plain round bars under repeated loading. According to his report reinforced concrete beams, because of incremental deformations, can become unserviceable under loads smaller than those which produce instantaneous collapse. Five

cycles of near ultimate loading were found to increase the deflection of the beam to 10 times its initial deflection.

As reported by Ruiz and George (3), "Repeated loads were found to increase the total deformation of the beams reinforced with deformed bars. The observed incremental deformations originated mainly in the tension zone and are believed to be caused by loss of bond and additional cracking or both, at the level of reinforcement. Neither the rotation nor the load carrying capacity of a reinforced concrete beams is affected by several (5-26) cycles of near ultimate loading when deformed bars with short yield plateau are used. Repeated loads at any load level cause progressive limited deformation in consecutive cycles for R.C. beams with deformed bars. This increase results mainly from the destruction of bond and from additional cracking at the level of reinforcement".

Victor D,J. and Ramamuorthy, K (7) carried several tests on bond resistance of different type of high strength deformed bars. Hinged beam type of specimen which will be described later had been used by them for bond test.

1.8 SCOPE OF INVESTIGATION AND PROBLEM FORMULATION :

In general, structures are subjected to live loads

and over loads which are time dependent phenomenon. The loads are repetitive in nature. In laboratory this type of load can be simulated by applying variable pulsating load at certain frequency by pulsator. The variable pulsating load consists of three bounds :

(1) The lower limit corresponding to some fraction of the ultimate load which represents the permanent load of the structure, the majority of which is the dead load of the structure.

(2) The upper limit corresponds to an assumed normal live plus dead load of the structure.

(3) The second upper limit called Peak load which can be applied less frequently than the first normal live load represents the combination of D.L, L.L. and W.L. or Earthquake load. In other words, Peak load represents combination of dead load and overloads.

A structure suffers elastic or plastic deformations when subjected to a variable pulsating load and there are every possibilities of failure due to fatigue or overstress condition at a load lower than the ultimate load capacity of the structure. Hence load factor based on static

strength should be checked to ensure safety and serviceability under variable pulsating load.

Very little work has been done on the field of high strength deformed bars ordinary and welded as reinforcement under variable pulsating load. In the present investigation, an attempt is made to explain the effect of variable pulsating loads on R.C. beams reinforced with high yield strength deformed bars. The problem can be stated as follows :

(1) Determination of strength properties of basic materials like concrete, HSD bars.

(2) Study of the tensile behaviour of HSD bars,ordinary and welded,under static and pulsating load.

(3) Study of the flexural behaviour of under reinforced concrete beams with ordinary and welded HSD bars. The details of the type of pulsating load and the type of specimen selected for the study, are explained in Chapter II.

CHAPTER II

EXPERIMENTAL INVESTIGATION

2.1 INTRODUCTION :

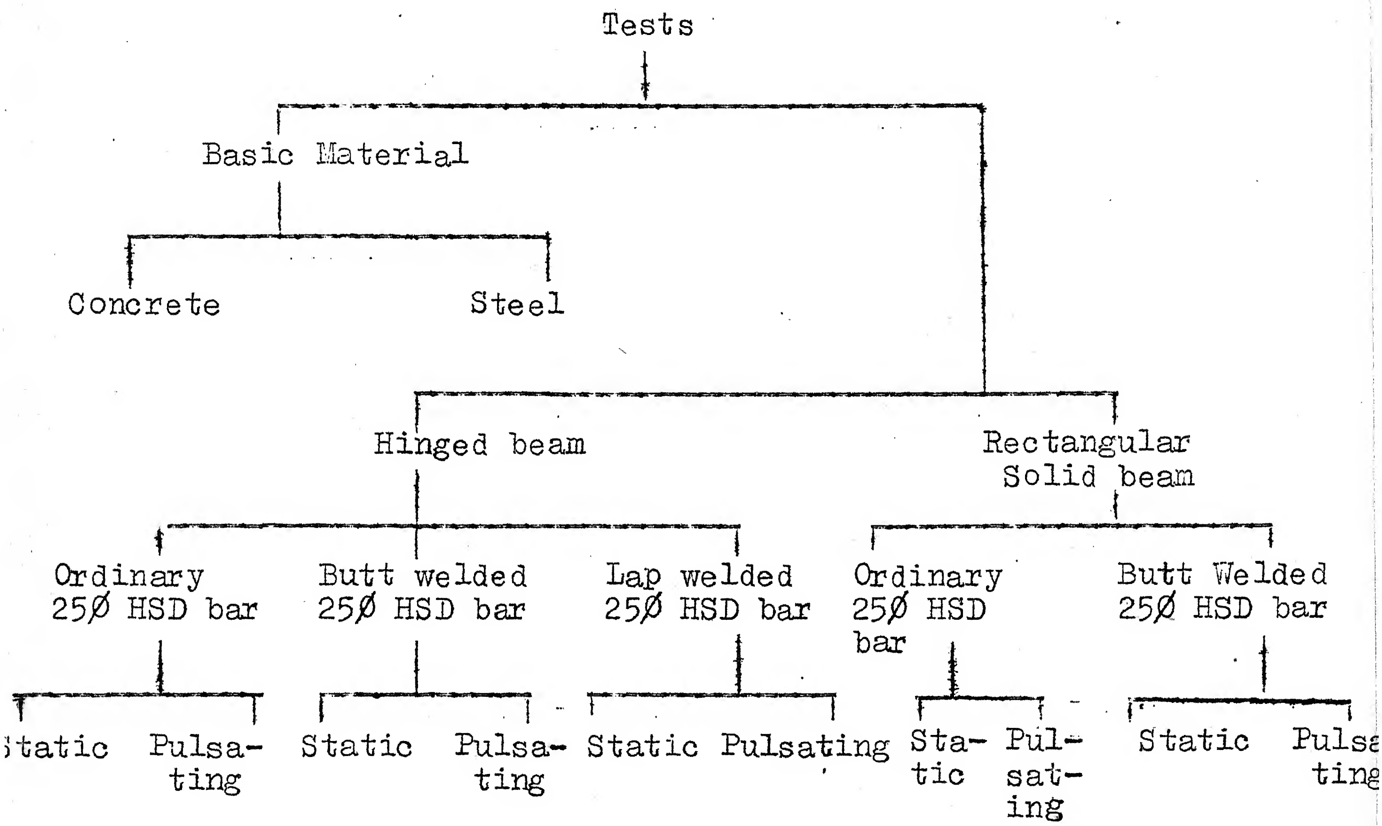
The purpose of the present experimental investigation was to find out how far high strength deformed bars, with and without weld, are safe under variable pulsating loads, when used as reinforcement in R.C. Structures. Also an attempt is made to check the adequacy of the load factors specified by Indian Standard.

For higher diameter bars, bond or lap length is higher as compared to lower diameter bars and hence there is always some wastage of reinforcing material. The space for laying concrete is thus reduced by providing higher lap lengths or bond lengths. For smaller diameter bars, these are not so severe but cost of welding is more. Hence in this study, 25 mm diameter high strength deformed bars were chosen.

Two types of welds (1) Butt weld (2) Lap weld with longitudinal beads can be done in reinforcements. The welding procedure and the precautions taken are discussed in the following section.

Ordinary and welded bars were tested in direct tension under static and pulsating load. Also reinforced concrete beams reinforced with and without welding in bars were tested in flexure under static and pulsating loads.

2.2 LIST OF TESTS :



2.3 TEST SPECIMEN :

2.3.1 DESIGN OF SPECIMENS :

The specimens selected for investigation were of two types:

(1) Hinged beam (hinge at the centre) with 25 mm dia. high yield strength deformed bar as shown in Fig. 2.1. This type of beam was selected to produce direct tension on the bar.

(2) Under reinforced concrete rectangular beam with high yield strength deformed 25 mm dia. bar as tensile reinforcement as shown in Fig. 2.2.

A two point loading as shown in Fig. 2.1 and Fig. 2.2 is adopted to ensure a pure bending failure of the specimens. The cross-sectional and overall dimensions are selected so that (1) the specimens can be easily handled and (2) the load carrying capacity of the specimens will be within the capacity of the hydraulic jacks available in the laboratory. The specimens were designed so as not to fail in shear while reaching an ultimate moment capacity.

In all the hinged beams the reinforcements were same. Ordinary, butt welded and lap welded 25 ∅ HSD bars were provided at the bottom of hinged beams such that direct tension will be developed on HSD bar when subjected to a two point loading. Similarly for the solid rectangular beams all the reinforcements were same except the type (with weld or without weld) of 25 ∅ HSD bar as tensile reinforcement.

Bending moment developed in the hinged beam under two point loading produced compression in the hinge and tension in the 25 Ø HSD bar (discussed in Appendix A). The variable in all the specimen was the type of joint (ordinary i.e., without any welded joint, butt welded joint and lap welded joint) in the 25 Ø HSD bar. Welded joints were provided at the middle of the specimens. Details of the specimens and their reinforcements etc. are as shown in Figs.2.4, 2.5 and Table 2.1. Specimen calculations for the design of test specimens are given in Appendix 'A'.

2.3.2 BASIC MATERIALS :

The specimens were cast with concrete consisting of granite of 12.5 mm down, Kalpi sand of F.M. = 2.7 and ordinary portland cement. Concrete mix was used 1:0.9:1.8 by weight, with an average water cement ratio of 0.42. Average slump was 2 cm to 2.5 cm.

25 mm. dia high yield strength deformed bars were tested in Universal Testing Machine. Stress-strain curve based on three experiments of high yield strength deformed 25 mm. dia. bar is given in Fig. 2.13. Stress-strain curve for repeated loading upto two cycle is shown in Fig. 2.14.

2.4 CASTING AND CURING :

2.4.1 WELDING OF BARS :

The bar ends to be welded for butt joint were shaped and cleaned from rust. The angle between bevel faces were kept 60°. A small gap of 2 to 3 mm was kept between bevel faces for root run. Bars were aligned and kept in proper axis before the root run in order to minimize crookedness in bar after welding. The sequence of run is shown in Fig. 2.3. After four runs the bar was turned by 180° and welding continued. When the last run (9), as shown in Fig.2.3 was deposited, the bar was gradually rotated by a full circle. The diameter of the weld was 1.2 times bar diameter, i.e. 30 mm. Lap weld joints were done according to the I.S. Code.

2.4.2 MAKING OF MOULDS :

Two wooden moulds were prepared as shown in Fig. 2.6. At the time of casting of simple rectangular beams those cutouts were repaired to get same cross-section throughout the length.

2.4.3 MAKING OF HINGE :

The hinge arrangement consisting of two V-blocks

and a 50 mm. dia roller was made. Holes were made on V-blocks to fix the hinge on the struts embeded in the specimen. The hinge arrangement is shown in Figs.2.7, 2.8 and 2.9.

2.4.4 CASTING OF THE SPECIMEN :

Two plates (10 mm thick) of size 150 mm x 196 mm were drilled and two struts on each plate were welded as shown in Fig. 2.8. The mould was kept on a levelled platform. For hinged beams, the hinge arrangement fixed with two plates on either side was kept in the proper position and then reinforcement cages were positioned. Materials were mixed in two batches for five minutes duration in an electrically driven mixer. The concrete was placed in the moulds in small quantities and thoroughly vibrated with a needle vibrator. Control specimens consisting of six 150 mm cubes were cast simultaneously. The top surface of the specimen and the cubes were made level and identification mark was given. For simple rectangular beams the reinforcement cage was positioned and casting was done in the same way as stated above.

2.4.5 CURING :

After 24 hours of casting the mould was taken out

from the specimen. The cubes were taken out from the mould and were placed in the curing bed. The hinge arrangement was taken out from the specimen leaving these plates with struts embeded in the concrete. Then the specimen was covered with damp gunny bags. The gunny bags were kept damp for the rest 27 days. Three cubes were tested after 28 days and the remaining on the day of testing of the specimen. Casting schedule and cube test results are given in Table 2.1.

2.5 LOADING CYCLE :

Structures are subjected to dead load, live load and overload conditions. This type of loading was simulated in the laboratory by a remote control pulsator. Real life structures are subjected to a comparatively low frequency where as the experiment had to be conducted at such a frequency so as to minimize the time required.

In the present investigation, the specimens were subjected to a variable pulsating load by a pulsator. The lower load P_1 correspondingto permanent dead weight of the structure and upper load P_2 correspondingto the combination of dead load and normal design load, acts as the lower and upper limit of the pulsating load. The frequency of

application of pulsating load was chosen in the range of 690 to 710 cycles per minute.

After every 150 minutes upper load P_2 is raised to a peak load P_{peak} (corresponds to the combination of D.L, L.L. and W.L. or Earthquake load, or dead load and overload) within 1 minute and lowered to P_2.

The loading cycle adopted in this experimental investigation is shown in Fig. 2.10. According to this, the pulsating load which corresponds to the design live load (varying from lower limit P_1 to upper limit P_2) was applied continuously for about 5 hours in a day; and the overload P_{peak} (Peak load) was applied twice during loading at an interval of $2\frac{1}{2}$ hours. The load was then completely reduced to zero and the specimens were left free from all external loads for about 19 hours. The fatigue effect produced by the accelerated frequency of loading was minimized by allowing a relaxation period of 19 hours. Calculations of lower (P_1), upper (P_2) and peak load (P_{peak}) for different specimens are given in Appendix 'B'. Selected values of P_1, P_2, P_{peak} for different specimens are given in Table 2.2.

2.6 TEST SET UP AND INSTRUMENTATION :

2.6.1 TEST SET UP :

The general arrangement of test set up is shown in Fig. 2.11 and Fig. 2.12. The two point loading was done by two hydraulic jacks of static capacity 24000 lbs (10.866 tonnes) each connected to RIEHLE SC 10 PULSATOR. A frame was set up on the structural floor of the laboratory in which two hydraulic jacks were supported and spaced 50 cm apart. The specimens were supported on roller bearings on the structural floor as shown in Fig. 2.11 and Fig. 2.12. The effective span for all the specimen was 1900 mm. One end of the specimen was a hinged support and the other end was a simple roller support in order to allow horizontal movement of the specimen along one direction. The details of loading point is also shown in Fig. 2.11 and Fig. 2.12.

In order to prevent undesirable vibration of the test frame four tie rods (as shown in Fig. 2.16) were provided and were tightened frequently to remove vibration if any. Both static and pulsating load tests were performed on the same set up.

2.6.2 INSTRUMENTATION :

The instrumentation adopted in testing is as shown

in Fig. 2.11 and Fig. 2.12. It consists of a set of dial gauges of sensitivity 0.01 mm in order to measure the deflection. Positions of various dial gauges for the hinged beams are shown in Fig. 2.11 and for simple rectangular beams are shown in Fig. 2.12.

To measure the deflection of roller and 25 ∅ HSD bar of hinged beams two plastic scales were provided as shown in Fig. 2.15.

Strain gauges were fixed on the rectangular beams as shown in Fig. 2.17 to measure the dynamic strains by the help of dynamic strain recorder (ENCARDIO RITE). Due to machine trouble those results were not correct and is not presented here.

2.7 TESTING PROCEDURE :

2.7.1 TESTING PROGRAMME :

The testing programme included pure static test and pulsating load test for each type of specimen. Static test was proposed to find the static ultimate collapse load and load-deflection behaviour of the specimen with the idea of checking the theoritical prediction of the ultimate load and making use of this result to predict the ultimate

collapse load of the other specimen more accurately. The other specimen was intended to study the behaviour under the designed pulsating and peak loads.

2.7.2 PREPARATION FOR TESTING AND INSTRUMENTATION :

The specimens were moved to the test bed and centre-lines of the specimens were clearly marked. For the hinged beams, the hinge arrangement was fixed on the plates (embeded in concrete) and then moved to the test bed. On those lines the positions of various dial gauge points were marked. The supports were fixed and the specimen was properly positioned and levelled with respect to the hydraulic jacks. Then deflection dial-gauges and plastic scales were fixed from rigid support in proper positions. Two straps on either side of the hinge were provided as shown in Figs. 2.11 and 2.18 to prevent splitting of concrete below the HSD bar. Specimens ready for testing are shown in Figs. 2.11, 2.12 and 2.17.

2.7.3 STATIC TEST :

After placing the specimen in proper position, the necessary instrumentation was done as explained in the previous section. An initial load of about 3% of jack capacity (5% of ultimate load) was applied and released

completely. After sometime zero reference readings of the dial gauges and scales were recorded. Also distance between roller centre and bar was measured. Then the specimen was subjected to monotonically increasing load till failure, in increments of 5% of jack capacity. At each increment dial gauge readings and scale readings were recorded. Cracks were also marked.

2.7.4 STATIC AND PULSATING LOADS TESTS :

Specimens A25, B25, B25 (2nd), C25, D25, E25, and E25 (2nd) were subjected to variable pulsating type of load and the test procedure adopted for all these specimens was same. Only the load ranges (P_1, P_2 and P_{peak}) were different depending upon cube strength and different load factor. Selection of load ranges are discussed in Appendix 'B'.

2.7.4 (a) STATIC TEST :

The necessary instrumentation was done after placing the specimen in proper position and an initial load of about 3% of jack capacity (2% of ultimate load) was applied and released to zero. After sometime, zero reference of the dial gauge readings, scale readings and the distance between roller and bar were recorded. Afterwards, load was applied in increments of 5% of jack capacity upto maximum load range corresponding to P_{peak} and then gradually released to zero. After five minutes, load was again applied in increments of 5% of jack capacity to the upper limit

(corresponding to P_2)and then released to lower limit(corresponding to P_1)and then again increased to P_2.

Cracks were marked as and when they appeared and progressed. The time interval between load increments was two to four minutes. Dial gauge readings (scale readings for hinged beams) were recorded at each load increment. This was done to know the behaviour of the specimen under cyclic loading since this phenomenon is similar to that of the pulsating loads except for the difference in the time of application of the loads and to extrapolate P_u by comparing the load-deflection curve of this specimen with that of the previous specimen.

2.7.4 (b) PULSATING LOAD TEST :

In the morning, dial gauge readings (also scale readings for hinged beams) at no load and upper load (P_2) were recorded. Then pulsating load varying between P_1 and P_2 with a frequency of 700 CPM was applied and the pulsator was run for five hours continuously. Just after 150 minutes and 300 minutes, the upper load (P_2) was increased to P_{peak} and then reduced to the upper load level (P_2) in two minutes. During this pulsating load and also at the times of applying peak load, the behaviour of the

specimen and crack propagation was observed. After 5 hours when the second peak load was applied and reduced to upper load (P_2), the pulsating load was changed to a static one and the load was maintained constant at the upper load level (P_2) and the dial gauge readings (scale readings also for hinged beams) were recorded. Then the load was completely released to zero and the zero load dial gauge readings (also scale readings for hinged beams) were recorded after some time.

The pulsating and peak loads applied in a day, as described above (shown in Fig. 2.10), which constitutes a loading cycle, was repeated on the next days and necessary readings were recorded.

Dial gauge readings at no load and upper load were recorded during dynamic test to study (i) the cumulative damage due to pulsating and peak loads and (ii) the creep recovery during the relaxation period.

2.7.4 (c) POST-PULSATING STATIC TEST :

After one million cycles, the specimen D25 was tested under monotonically increasing load till failure, in order to know the ultimate load after the pulsating load

programme.

2.8 SELECTION OF LOAD LEVELS :

In the variable pulsating load test lower load (P_1), upper load (P_2) and peak load (P_{peak}) were selected on the basis of load factors.

Static tests were done to know the ultimate strength and load deflection behaviour. From the result of static test and cube strength , ultimate load of the specimen (to be tested under pulsating load) was anticipated by interpolation or extrapolation. The interpolation or extrapolation was done both from strength and deflection and is discussed in Appendix 'B'. From the anticipated ultimate load the load levels were found out using the load factors specified by IS 456-1968 and IS 456 - (draft 1978). Depending upon the previous test result selection of load levels was done and is presented in Appendix B.

AND CUBE RESULTS

(25∅ HSD bar)

Sl. No.	Type	Specimen No.	Type of Bar	Reinforcements	Date of Casting	Date of Testing	28 Days Average Cube Strength (kg/cm^2)	Age on test date	Average cubic comp. Strength at test date(kg/cm^2)
1	Hinged Beam	SA25	Without Weld	3 Nos.10∅ as comp. reinforcement and 3 Nos.10∅ as tensile reinforcements as shown in Fig. 2.4	8/3/78	10/4/78	410	33	450
2		A25	-do-		9/3/78	12/4/78	407	34	420
3		SB25	Butt welded		13/3/78	22/4/78	402	40	430
4		B25	-do-		15/3/78	26/4/78	406	42	465
5		B25 (2nd)	-do-		16/3/78	3/5/78	388	48	415
6		SC25	Lap welded		17/3/78	20/5/78	398	64	445
7		C25	-do-		18/3/78	25/5/78	390	68	480
8	Solid Rectangular beam	SD25	Without Weld	3 Nos.10∅ as Comp.reinforcement.1 No.25∅ bar as tensile reinforcement as shown in Fig. 2.5	3/5/78	2/6/78	360	30	377
9		D25	-do-		6/5/78	15/6/78	350	40	360
10		E25	Butt Welded		16/5/78	26/6/78	296	41	302
11		E25 (2nd)	-do-		18/5/78	1/7/78	298	44	306

NOTE: 6 Cubes were taken for each specimen

TABLE 2.2 : LOAD LEVELS

Specimen No.	P_u^T tonnes	P_u Anticipated tonnes	P_u^E tonnes	Lower Load P_1			Upper Load P_2			Peak Load P_{peak}		
				In tonnes	Percent of P_u	Percent of Jack Capacity	In tonnes	Percent of P_u	Percent of Jack Capacity	In Tonnes	Percent of P_u	Percent of Jack Capacity
SA25	-	-	4.89	STATIC TEST								
A25	-	4.89	-	1.08	22.0	10.0	3.27	66.8	30.0	3.91	80.0	36.0
SB25	-	-	5.43	STATIC TEST								
B25	-	5.43	-	1.08	20.0	10.0	3.10	57.0	28.5	4.10	75.5	37.75
B25 (2nd)	-	5.43	-	1.08 (1.08)	20.0 (20.0)	10.0 (10.0)	2.10 (2.82)	39.0 (52.0)	19.5 (26.0)	2.82 (4.16)	52.0 (76.6)	26.0 (38.25)
SC25	-	-	5.27	STATIC TEST								
C25	-	5.23	-	1.08	20.6	10.0	2.82	54.0	26.0	3.26	62.3	30.0
SD25	6.327	-	7.47	STATIC TEST								
D25	6.31	6.555	7.27	1.08 (1.08)	16.5 (16.5)	10.0 (10.0)	4.37 (3.91)	66.67 (59.65)	40.25 (36.0)	5.46 (4.89)	83.3 (74.6)	50.25 (45.0)
E25	5.723	6.194	-	1.08	17.4	10.0	4.13	66.67	38.0	5.16	83.3	47.5
E25 (2nd)	5.804	6.03	-	1.08	17.9	10.0	4.02	66.67	37.0	5.024	83.3	46.25

NOTE: - 1. Values inside bracket represents the new load level as discussed in Chapter III and Appendix B.

2. 10 Percent of Jack Capacity = 1.0863 tonnes

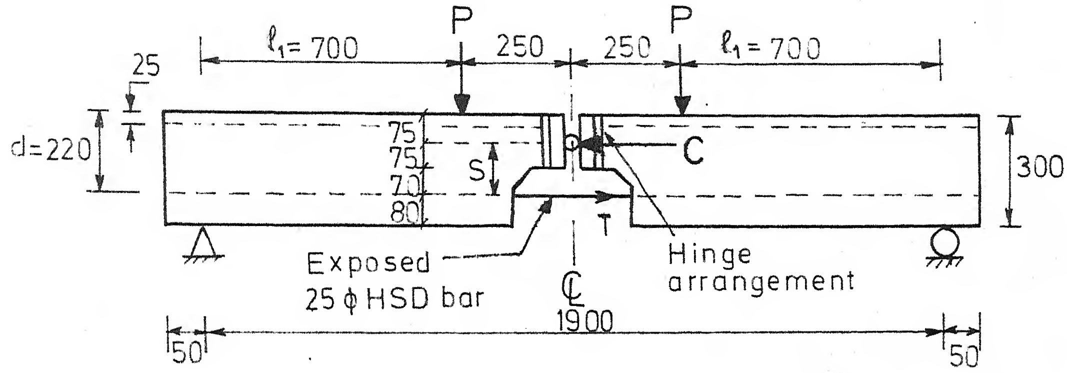


Fig. 2·1 Hinged beam and the load points

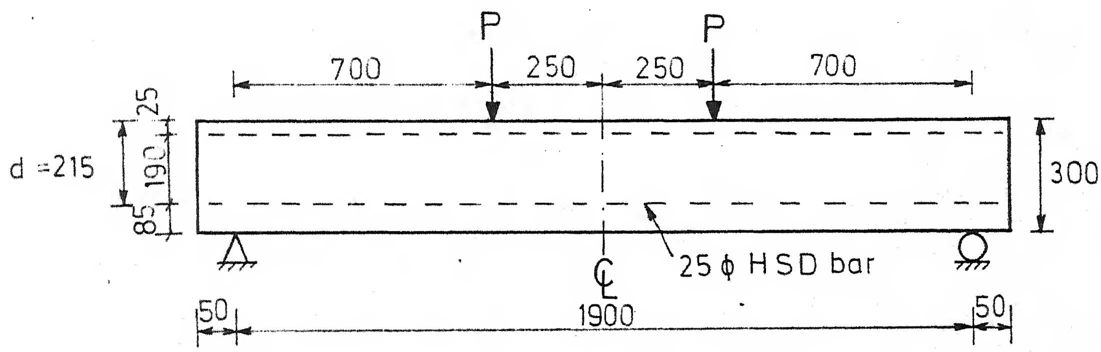


Fig. 2·2 Rectangular beam and the load points

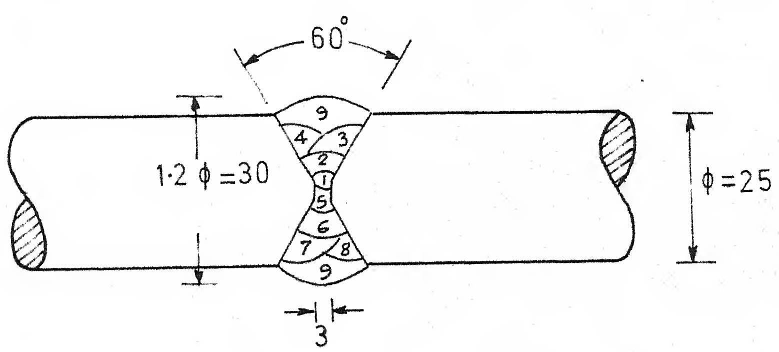


Fig. 2·3 Sequence of run

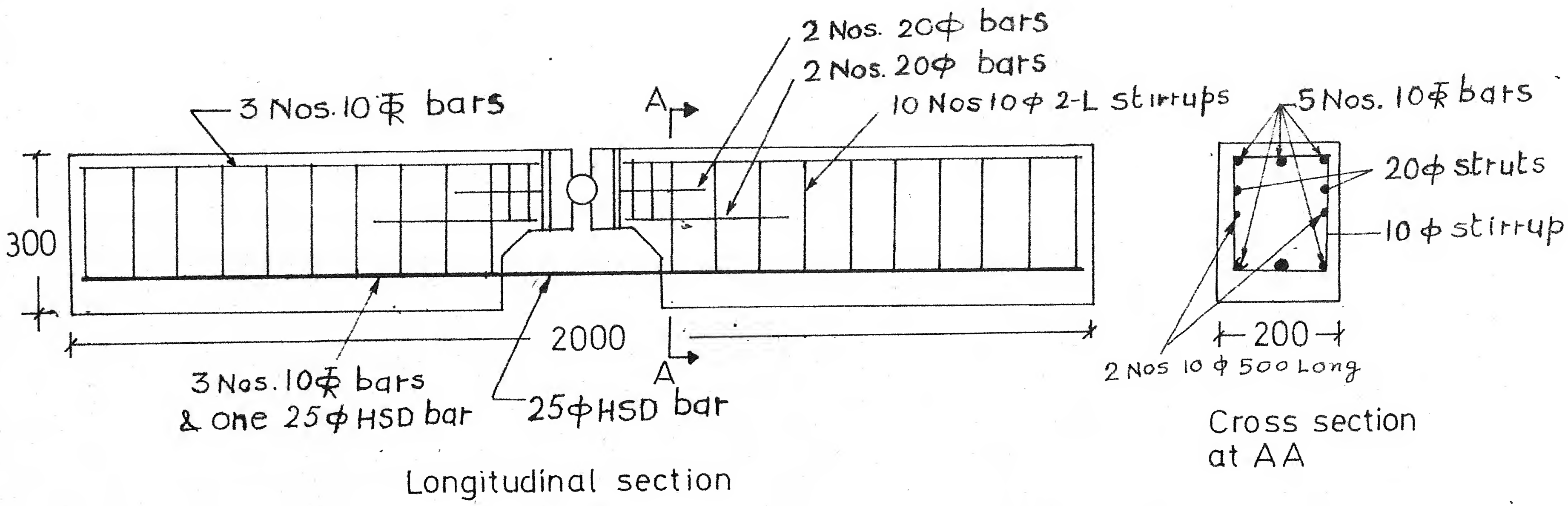


FIG. 2.4 REINFORCEMENT DETAILS OF HINGED BEAM

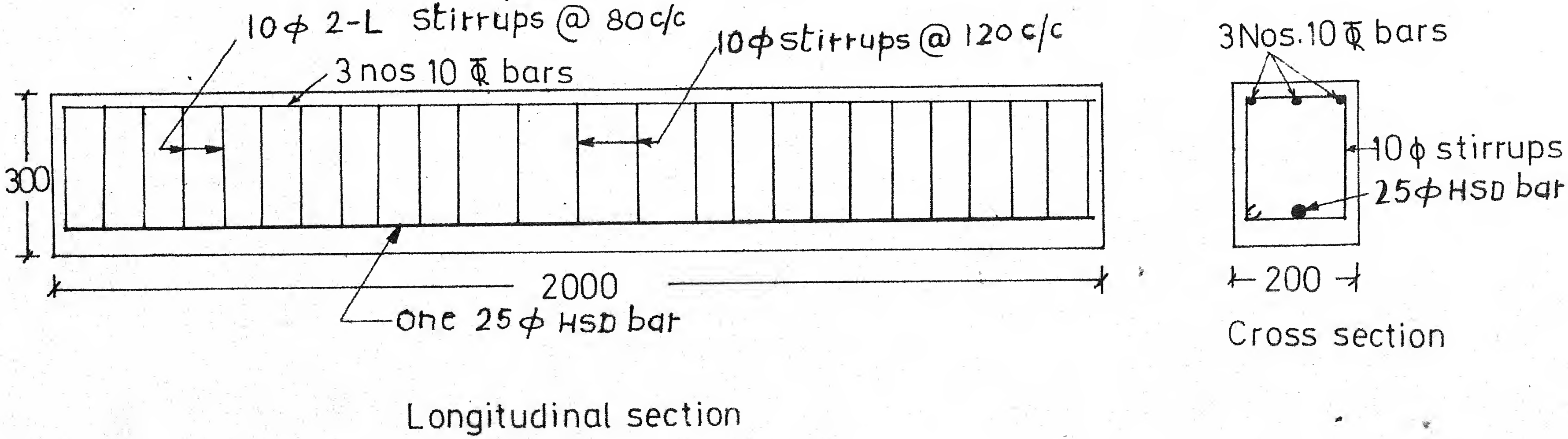


FIG. 2.5 REINFORCEMENT DETAILS OF RECTANGULAR BEAM

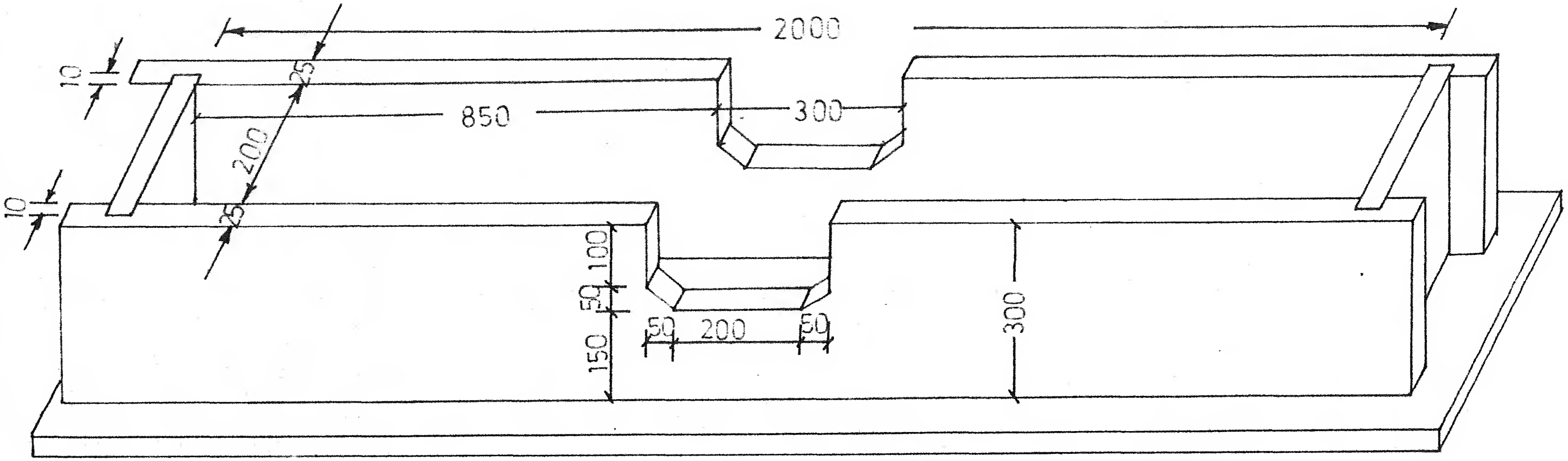


Fig. 2·6 Wooden mould for hinged beam

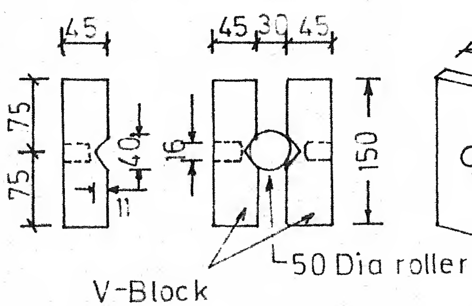


Fig.2·7 V-Block and hinge

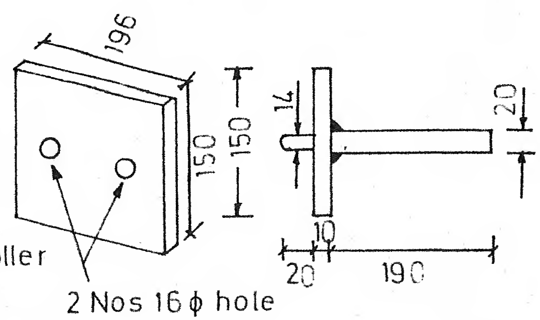


Fig.2·8 Plate with struts

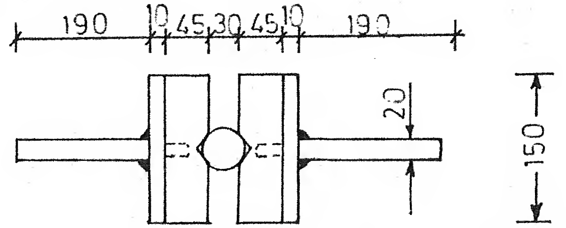


Fig.2·9 Total hinge arrangement

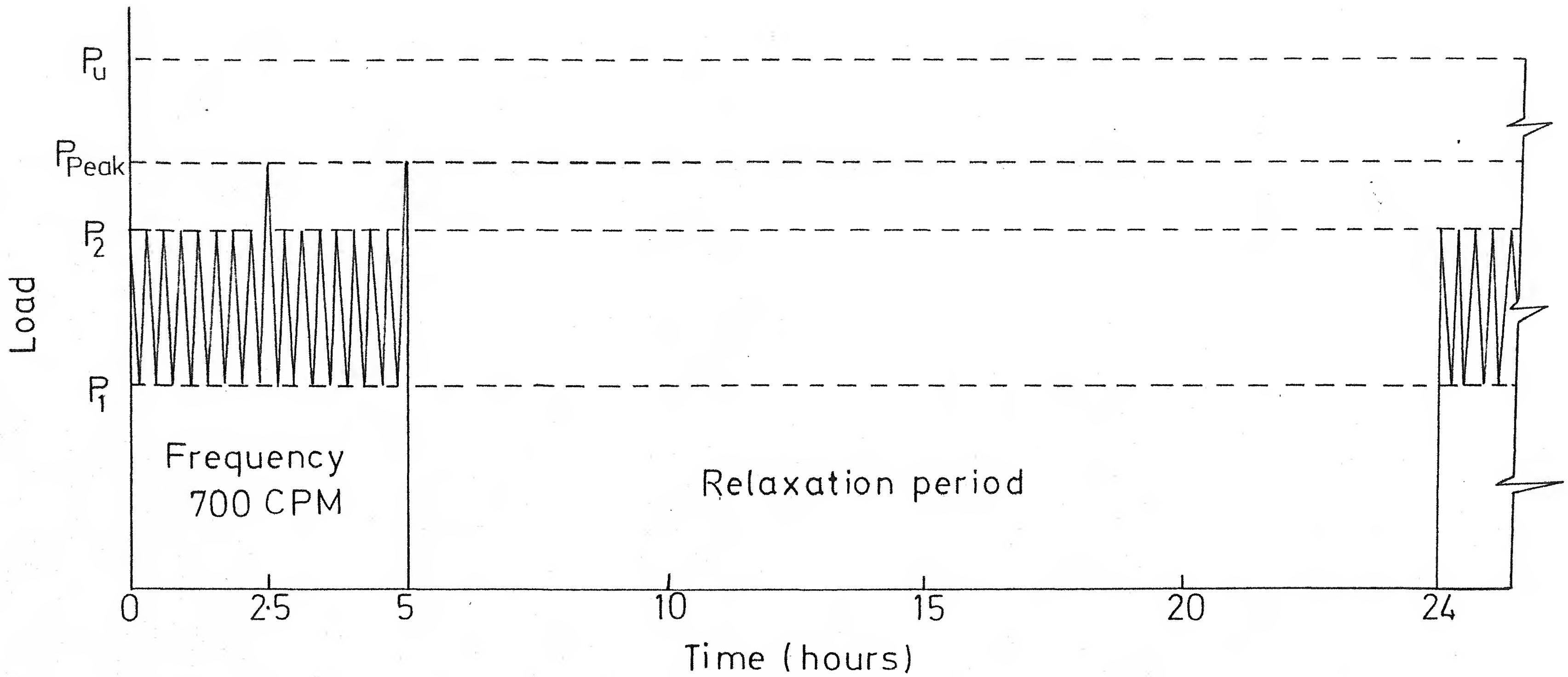


FIG. 2·10 LOADING CYCLE

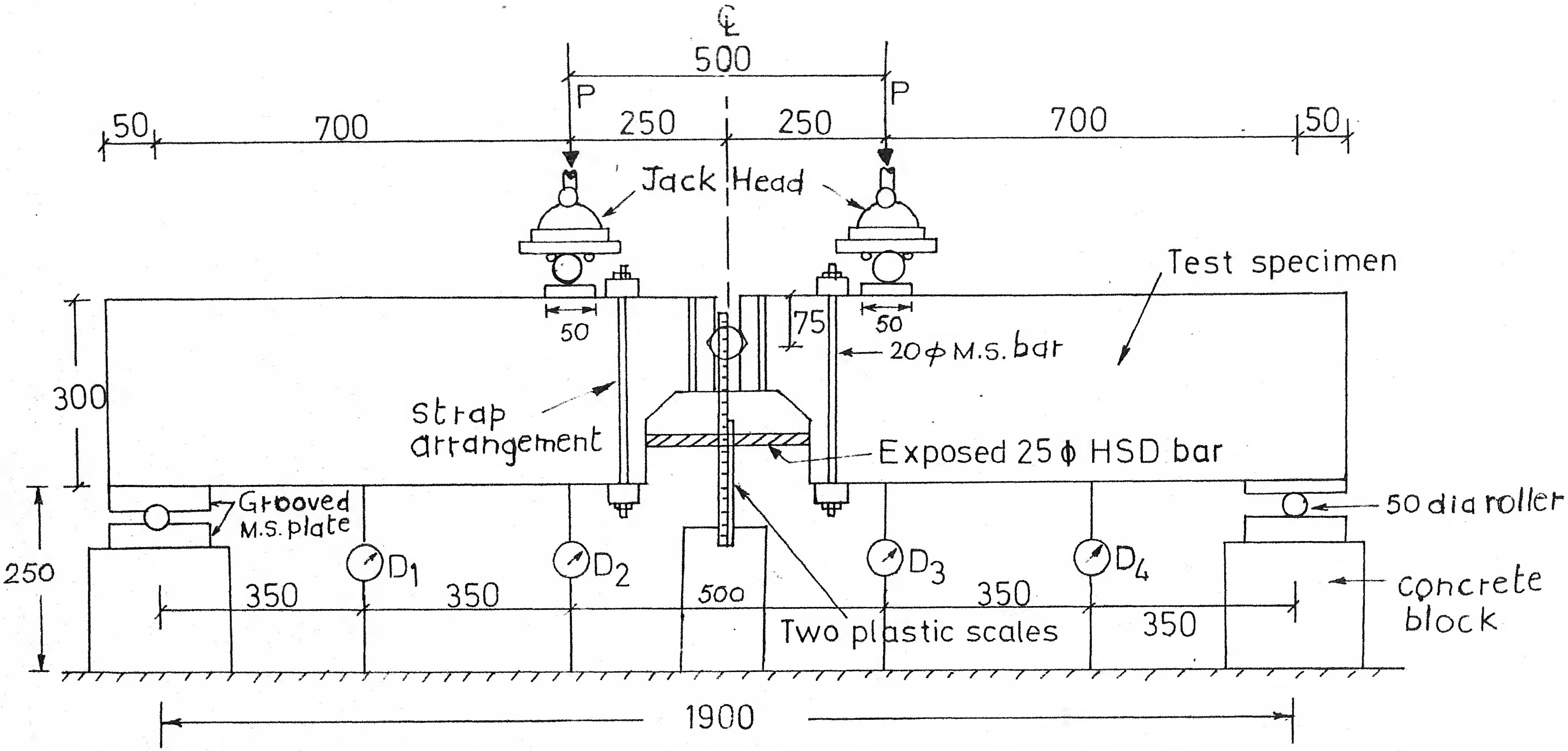


FIG. 2·11 SET UP AND INSTRUMENTATION FOR HINGED BEAM

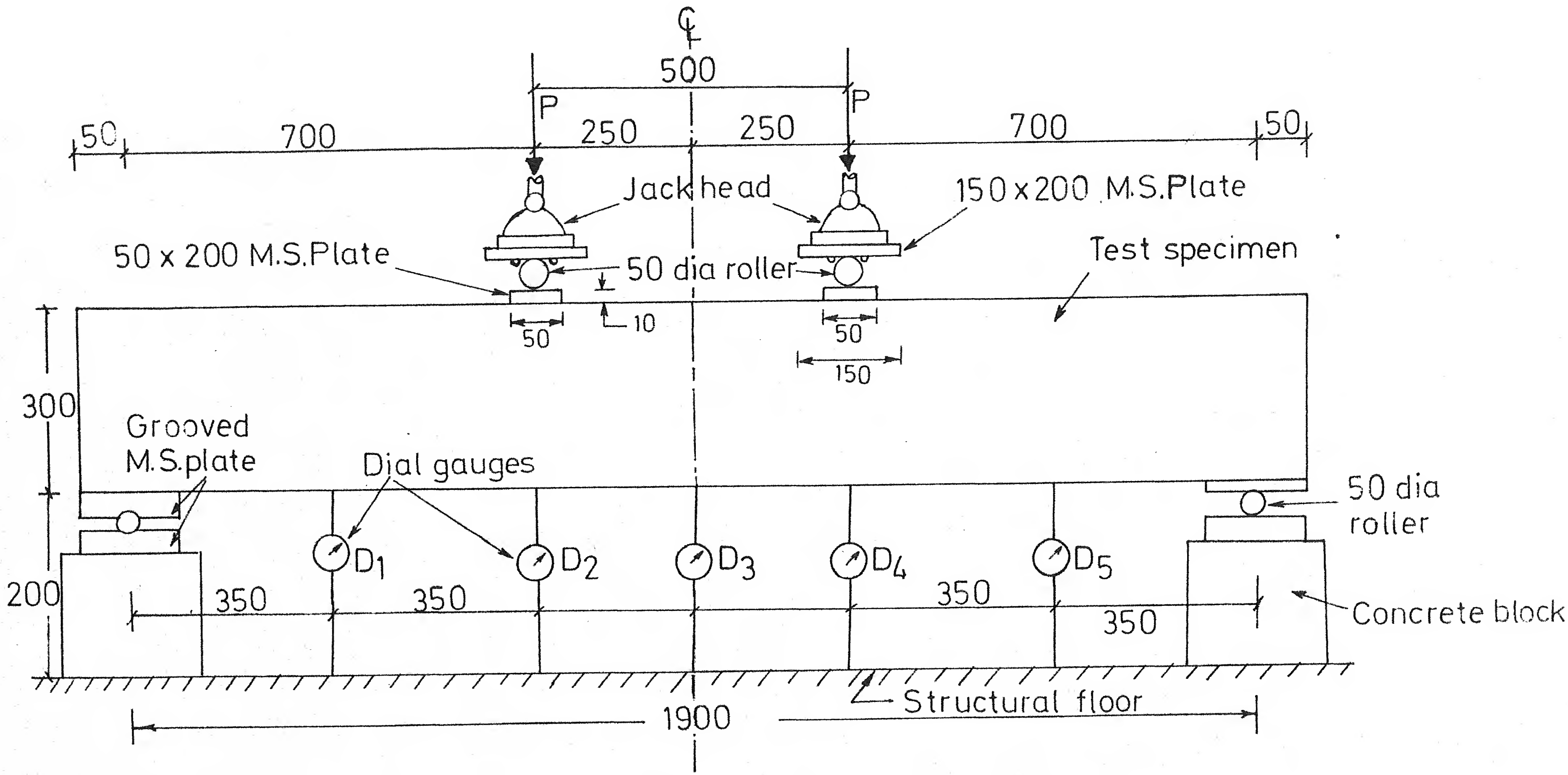


FIG. 2.12 SET UP AND INSTRUMENTATION FOR RECTANGULAR BEAM

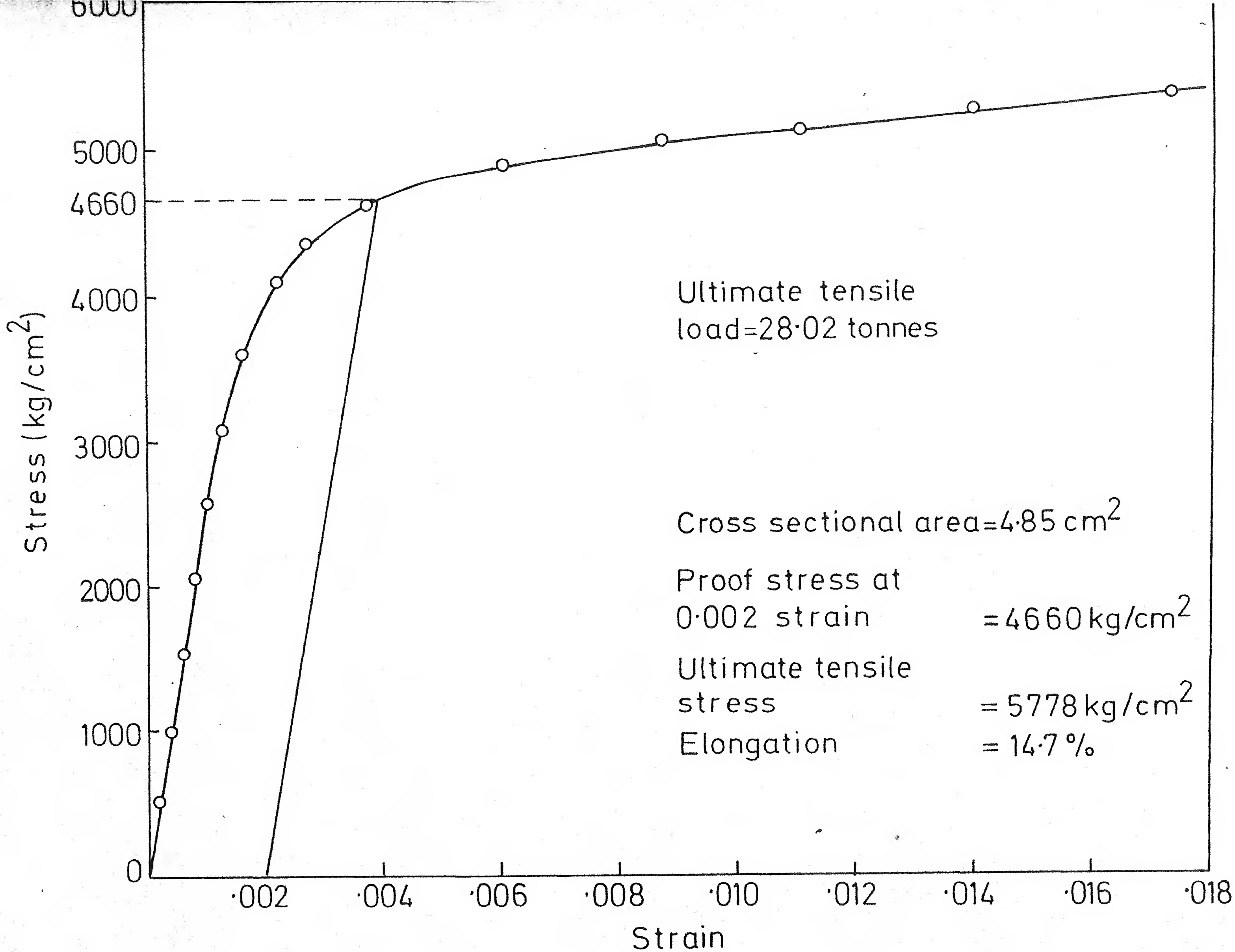


FIG. 2·13 STRESS STRAIN CURVE OF 25 ϕ HSD BAR BASED ON AVERAGE OF THREE TESTS

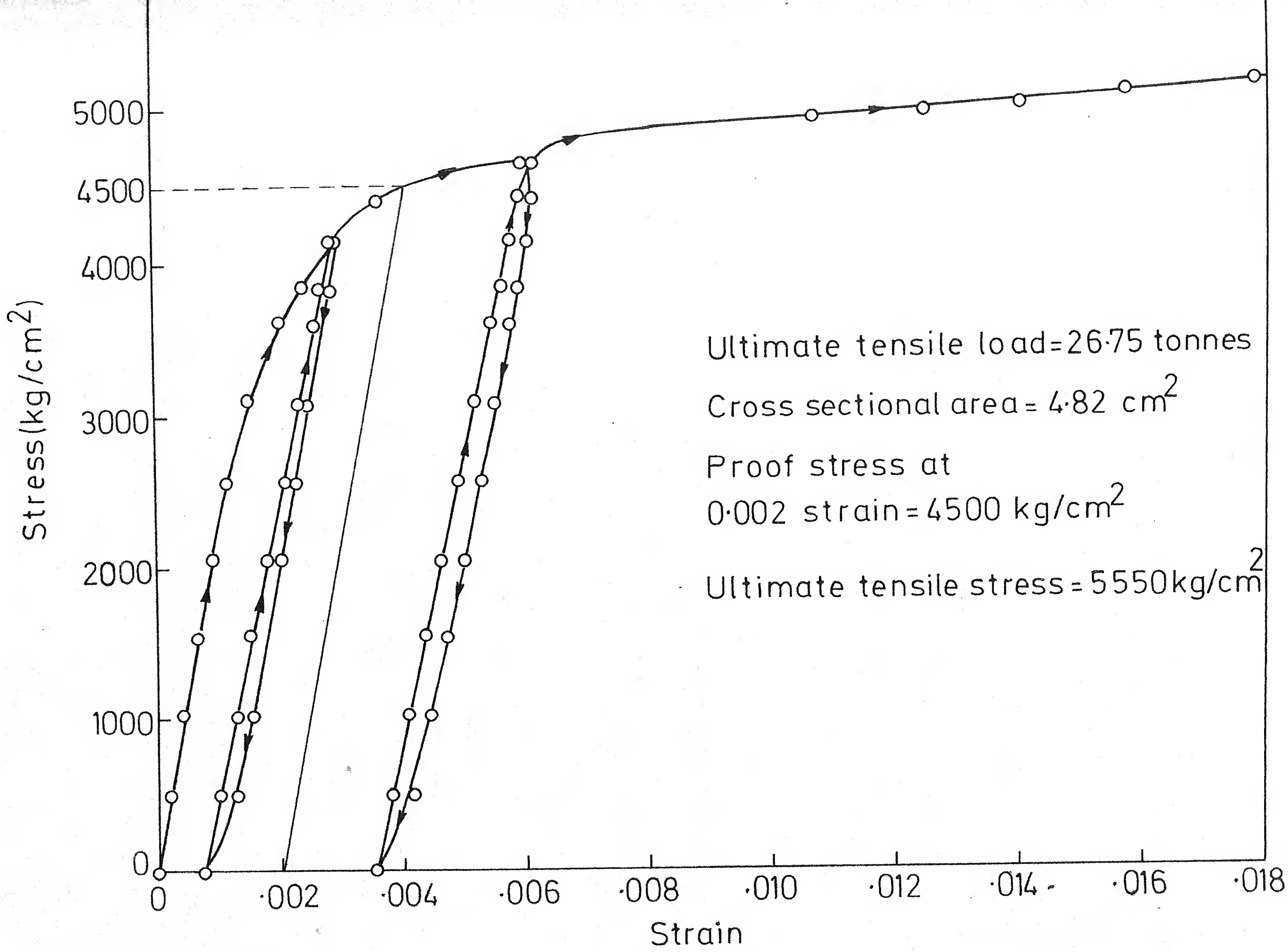


FIG. 2·14 STRESS-STRAIN CURVE OF 25 ϕ HSD BAR UNDER LOADING AND RELOADING

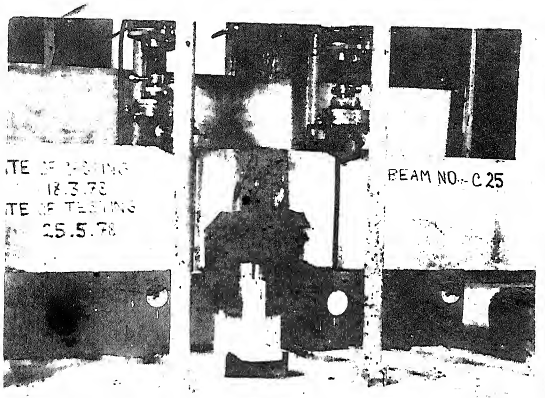


Fig. 2.15 Plastic scales to measure deflections of roller and 25 Ø HSD bar

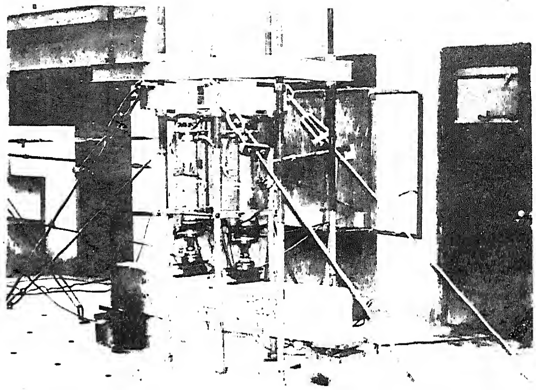


Fig. 2.16 Test frame and four tie rods

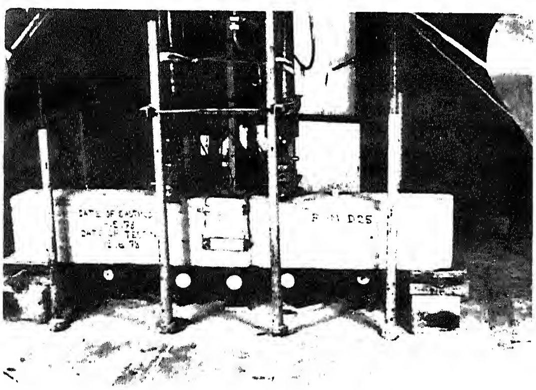


Fig. 2.17 Specimen D25 ready for testing

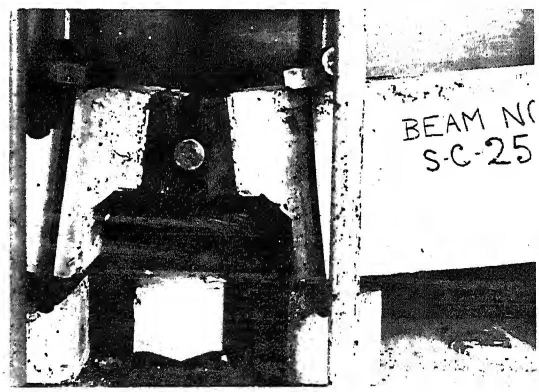


Fig. 2.18 Strap fixed on either side of the hinge

CHAPTER III

RESULTS AND DISCUSSIONS

3.1 INTRODUCTION :

High yield strength deformed 25 mm dia. bar was tested under monotonically increased tensile load. Proof and ultimate tensile stresses were found to be 4660 kg/cm^2 and 5778 kg/cm^2 respectively where as characteristic strength as per I.S. code is 4250 kg/cm^2 and 4950 kg/cm^2. When it was subjected to two cycles of repeated tensile load, proof stress and ultimate tensile stress was found to be 4550 kg/cm^2 and 5500 kg/cm^2.

Strength of specimens was determined on the basis of experimental strength of the HSD bar and from their individual cube strength results. Some of the specimens were tested under static load in order to know the variation of experimental ultimate strength against theoretically predicted ultimate strength. On the basis of static test results,ultimate strength of the specimens (to be tested under variable pulsating load) was anticipated by interpolation or extrapolation. The interpolation or extrapolation was done both from strength and deflection

criteria (discussed in Appendix B). From the anticipated strength,load levels of pulsating load were fixed using load factors.

Hinged beams SA25, SB25 and SC25 were tested under static load,till failure,to find the ultimate strength capacity. These were control beams and from their behaviour, ultimate strengths of hinged beams A25, B25 and B25 (2nd) were predicted to select the load levels for testing under variable pulsating load. Hinged beams A25, B25, B25 (2nd) and C25 were then tested under pulsating load. Various details of those seven specimens are shown in Tables 2.2 and 3.1. Stresses developed corresponding to different load levels were calculated (as shown in Appendix B) and are shown in Table 3.3.

Solid rectangular beam SD25 was tested under static load till failure to find the ultimate strength capacity and load deflection behaviour. It served as control beam to predict the ultimate strength capacity of beams D25, E25 and E25 (2nd) in order to select the load levels of variable pulsating load. Specimens D25, E25 and E25 (2nd) were then subjected to variable pulsating load. Various details of these four specimens (selection of load levels shown in Appendix B) are shown in Tables 2.2 and 3.2.

TABLE 3.1 : PULSATING LOAD TEST RESULTS OF HINGED BEAMS

Frequency = 700 C.P.M.

Specimen No.	P_u Antici-pated in tonnes	P_1 in tonnes	P_2 in tonnes	P_{peak} in tonnes	Date	N (pulsating) cummulative Mega Cycle	N (overload) Cumulative	Combined Load Factor for P_2	Combined Load Factor for P_{peak}
A25	4.89	1.08	3.27	3.91	12/4/78	0.118	1	1.5	1.25
					14/4/78	0.318	3		
					18/4/78	0.498	5		
					19/4/78	0.680	7		
B25	5.43	1.08	3.10	4.10	26/4/78	0.052	1	1.75	1.32
					27/4/78	0.258	2		
B25 (2nd)	5.43	1.08	2.12	2.82	3/5/78	0.235	2	2.56	1.92
					4/5/78	0.505	4		
					5/5/78	0.841	6		
					6/5/78	1.173	8		
					8/5/78	1.497	10		
					9/5/78	1.774	12		
					10/5/78	2.107	14		
					11/5/78	2.450	16		
	5.43	1.08	2.82	4.16	12/5/78	2.564	17	1.92	1.30
C25	5.23	1.08	2.82	3.26	26/5/78	0.080	0	1.85	1.60

* For selection for load levels see Appendix B

TABLE 3.2 : PULSATING LOAD TEST RESULTS OF SOLID RECTANGULAR BEAMS

Frequency = 700 C.P.M.

Specimen No.	P_u Anticipated in tonnes	P_1 in tonnes	P_2 in tonnes	P_{peak} in tonnes	Date	N (Pulsating) Cummulative Mega Cycle	N(over load) Cummulative	Combined Load Factor for P_2	C.L.F. for P_{peak}
D25	5.87	1.08	3.91	4.89	15/6/78	0.096	1	1.5	1.2
					16/6/78	0.157	2		
					17/6/78	0.335	4		
					19/6/78	0.508	5		
					20/6/78	0.658	7		
					21/6/78	0.883	9		
					22/6/78	1.000	10		
E25	6.194	1.08	4.13	5.16	26/6/78	0.202	2	1.5	1.2
					27/6/78	0.506	4		
E25 (2nd)	6.030	1.08	4.02	5.024	1/7/78	0.150	1	1.5	1.2
					3/7/78	0.208	1		

* For selection for load levels see Appendix B

TABLE 3.3 : STRESS LEVELS

Specimen No.	Stress Level σ_1 kg/cm^2	σ_2 kg/cm^2	σ_{peak} kg/cm^2	$\frac{\sigma_1}{\sigma^E_{yp}}$	$\frac{\sigma_2}{\sigma^E_{yp}}$	$\frac{\sigma_{peak}}{\sigma^E_{yp}}$	$\frac{\sigma_1}{\sigma^E_{ult}}$	$\frac{\sigma_2}{\sigma^E_{ult}}$	$\frac{\sigma_{peak}}{\sigma^E_{ult}}$	No. of cycles (Mega cycle)
A25	1181	3602	4334	0.253	0.773	0.930	0.200	0.620	0.75	0.680
B25	1146	3314	4400	0.246	0.710	0.940	0.198	0.570	0.76	0.258
B25 (2nd)	1163	2292	3060	0.250	0.492	0.660	0.200	0.400	0.53	2.430
Same Specimen subjected to new load range	1163	3060	4514	0.250	0.660	0.969	0.200	0.530	0.78	0.130
C25	1163	3095	3591	0.250	0.664	0.770	0.200	0.535	0.62	0.080

NOTE: σ^E_{yp} = 4660 kg/cm^2 and σ^E_{ult} = 5778 kg/cm^2 are experimental stresses as per Fig. 2.13.

σ_1, σ_2, and σ_{peak} are stresses developed corresponding to load levels P_1, P_2 and P_{peak} respectively.

3.2 STATIC AND PULSATING LOAD TEST RESULTS OF SPECIMENS SA25 AND A25

Specimens SA25 and A25 were identical with 25 Ø HSD bar without welded joint.

3.2.1 STATIC TEST :

Specimen SA25 was tested under monotonically increasing static load till failure and Load deflection behaviour is shown in Fig. 3.1. The specimen failed due to failure at the section near the middle of exposed bar.

3.2.2 VARIABLE PULSATING LOAD TEST :

Specimen A25 (companion beam of SA25) was tested under variable pulsating load. Load deflection behaviour during static loading and unloading (including peak load) is shown in Fig. 3.2. After 0.68 mega cycles the specimen failed due to failure of exposed bar almost near the middle zone.

3.3 STATIC AND PULSATING LOAD TEST RESULTS OF SPECIMENS SB25, B25 AND B25 (2nd) :

These three specimens were similar in all respect. Butt welded joints of the deformed bars were exactly at

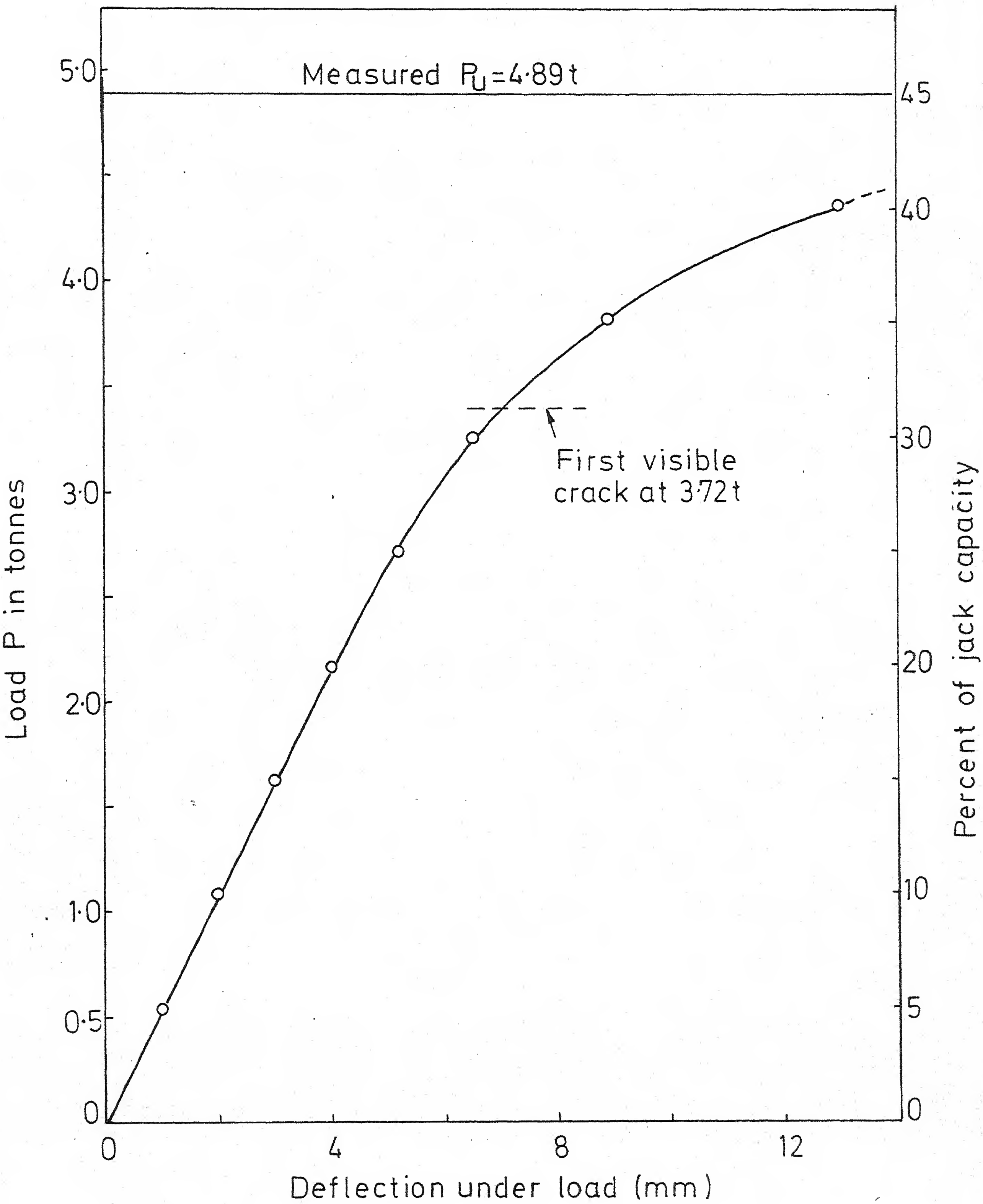


FIG.3·1 LOAD-DEFLECTION CURVE OF SPECIMEN SA 25

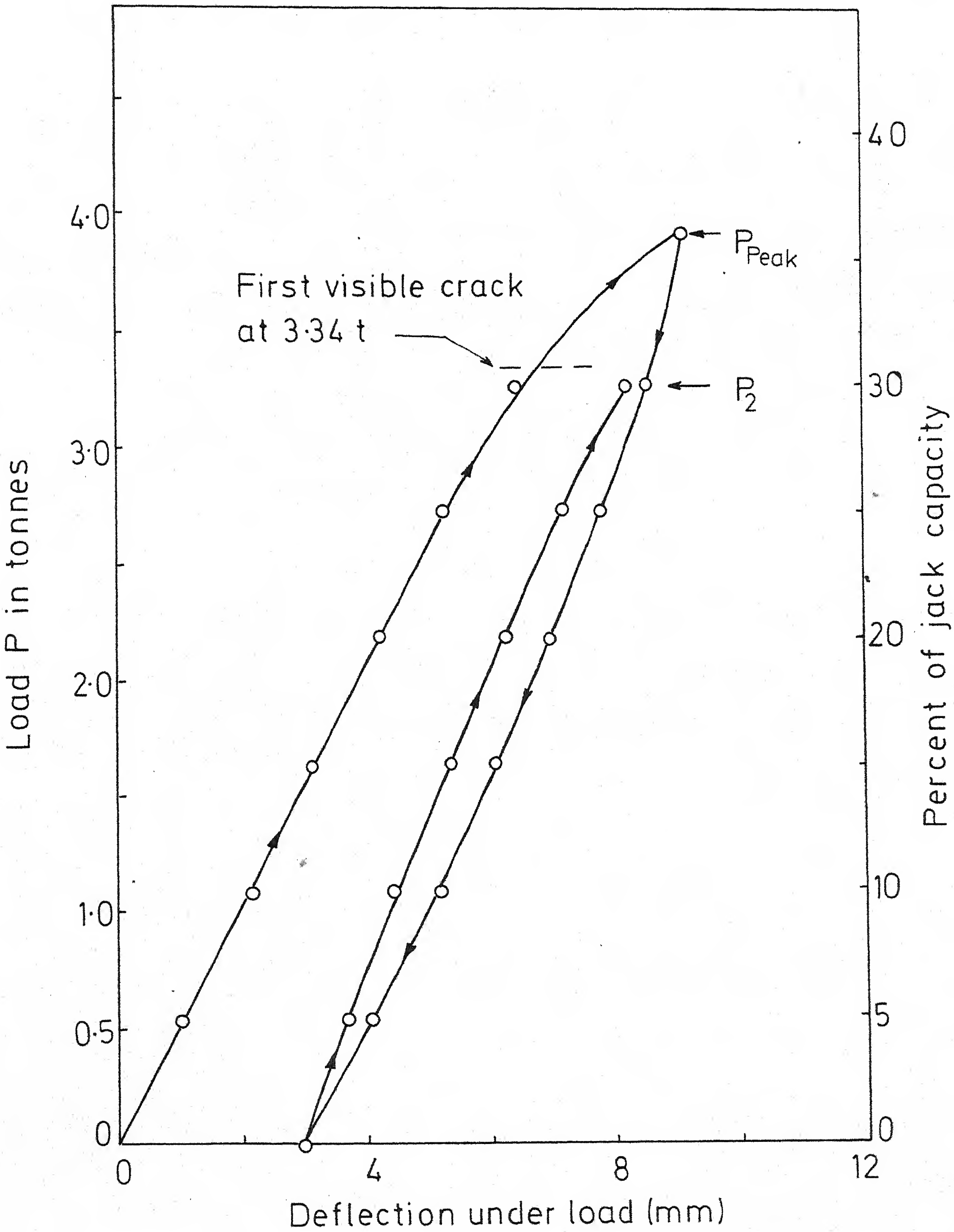


FIG. 3·2 LOAD-DEFLECTION CURVE OF SPECIMEN A25

the middle of the specimen.

3.3.1 STATIC TEST :

Specimen SB25 was tested under monotonically increasing static load till failure and load deflection behaviour is shown in Fig. 3.3. The specimen failed due to failure of 25 Ø HSD bar. The bar failed just after the welded joint.

3.3.2 VARIABLE PULSATING LOAD TEST :

Specimen B25 (companion beam of SB25) was tested under variable pulsating load. Load-deflection behaviour during static loading and unloading (including peak load) is shown in Fig. 3.4. After only 0.258 million cycles the specimen suddenly failed due to weld failure. The 25 Ø HSD bar failed through the welded joint. The failure plane was normal to the axis of the bar and is shown in Fig. 3.15.

Specimen B25 (2nd) was also tested under variable pulsating load under a lower load level compared to specimen B25. Load-deflection behaviour during static loading and unloading (including peak load) is shown in Fig. 3.5. The specimen did not fail upto 2.43 Mega cycles. Cumulative deflection Vs time at upper load level (P_2) and

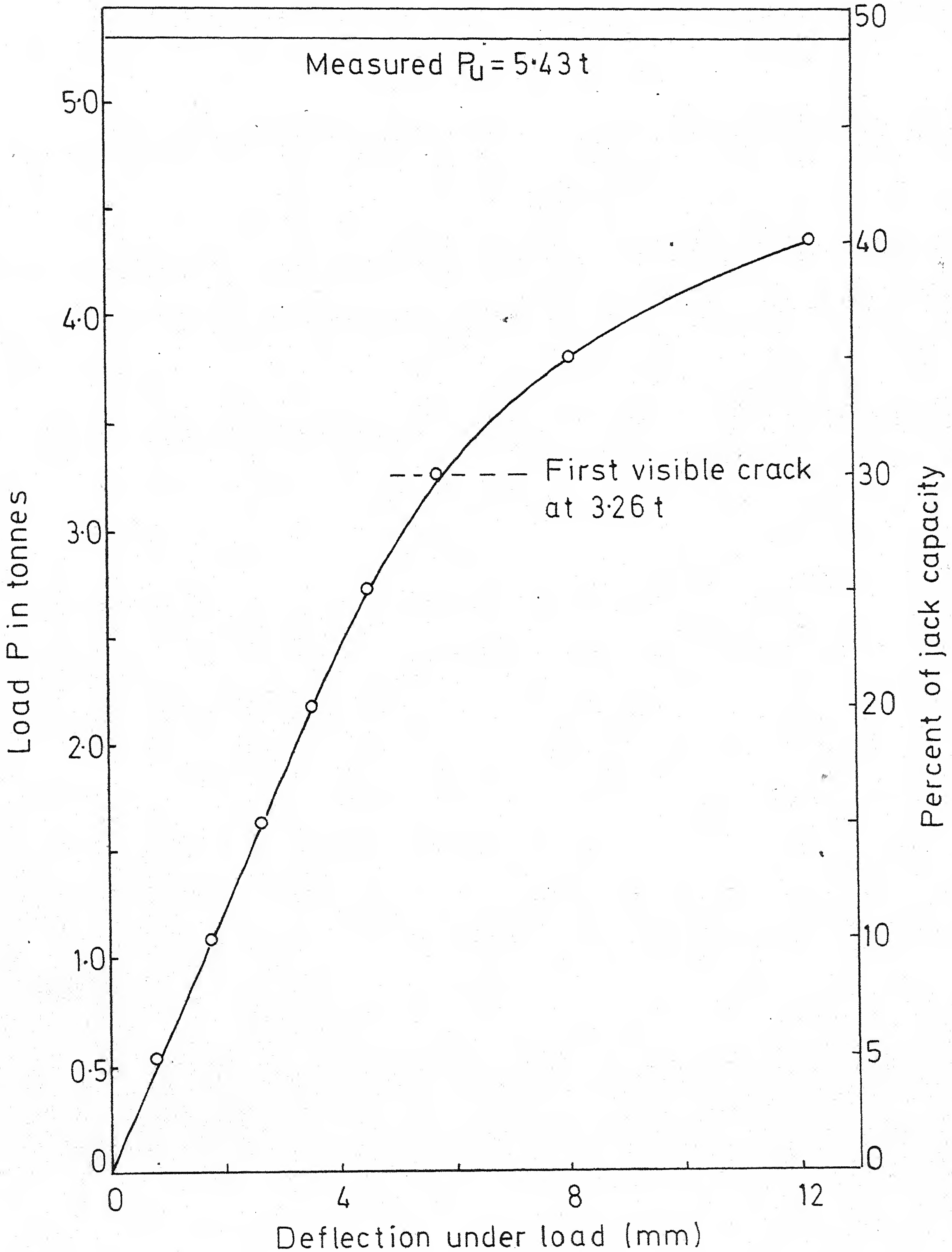


FIG. 3·3 LOAD-DEFLECTION CURVE OF SPECIMEN SB 25

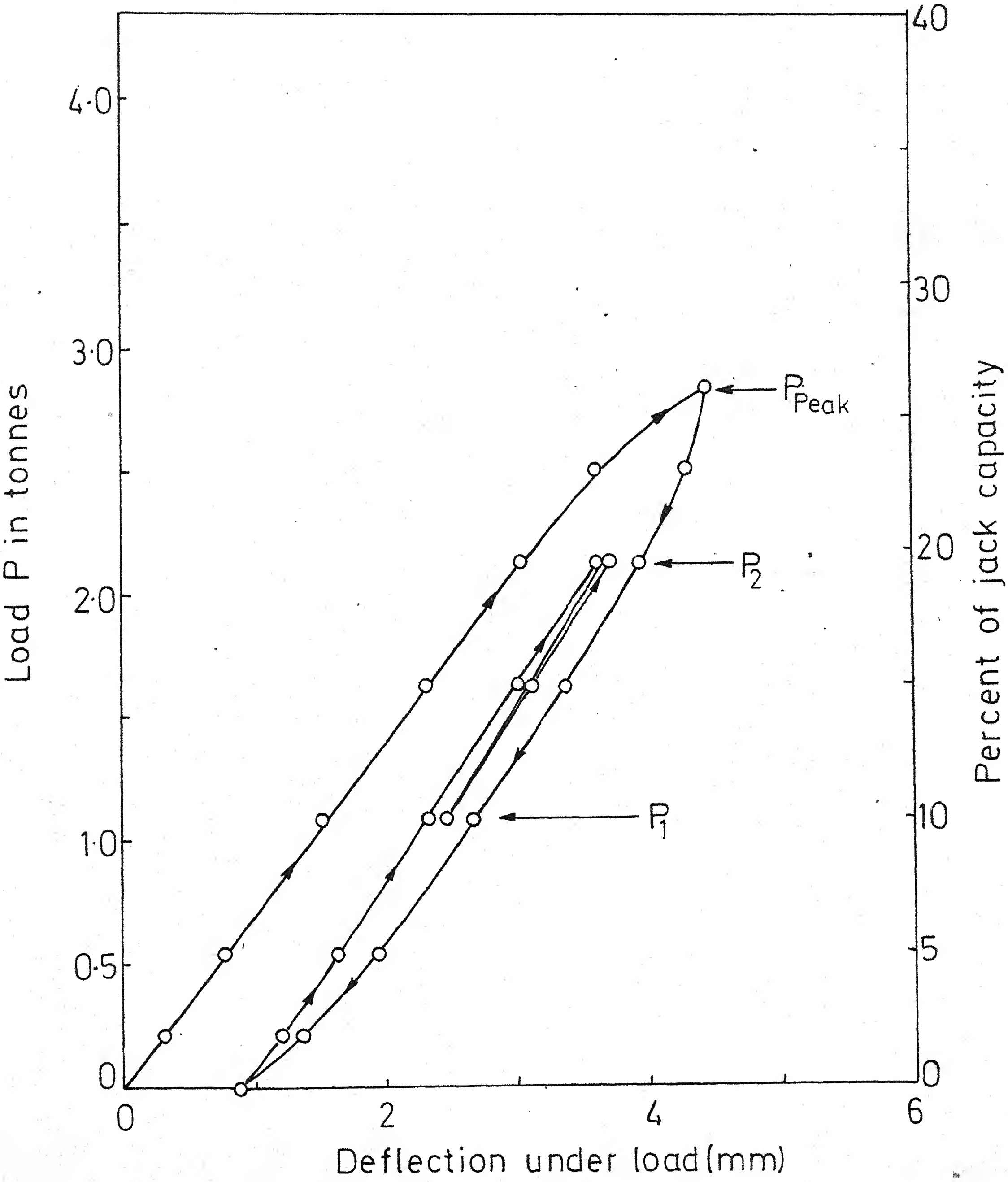


FIG.3.5 LOAD-DEFLECTION CURVE OF SPECIMEN B25 (2ND)

no load condition for the specimen is represented in Fig. 3.6.

After 2.43 million cycles the same specimen was subjected to variable pulsating load of different limits of load level. Those limits are listed below :-

P_1 = 1.08 t i.e; 10% of Jack capacity

P_2 = 2.82 t i.e; 26% of Jack capacity

P_{peak} = 4.16 t i.e; 38.25% of Jack capacity

With the new limits of variable pulsating load as stated above, the specimen failed after only 0.13 million cycles due to failure of the section next to the welded joint of 25 ∅ HSD bar. The weld failure was similar to the weld failure of specimen B25 as shown in Fig. 3.15. Within 0.13 million cycles the specimen suffered only one peak load level.

3.4 STATIC AND PULSATING LOAD TEST RESULTS OF SPECIMENS SC25 AND C25 :

These two specimens were identical both having lap welded joint of 25 ∅ HSD bars. The lap length was 250 mm.

3.4.1 STATIC TEST :

Specimen SC25 was tested under monotonically

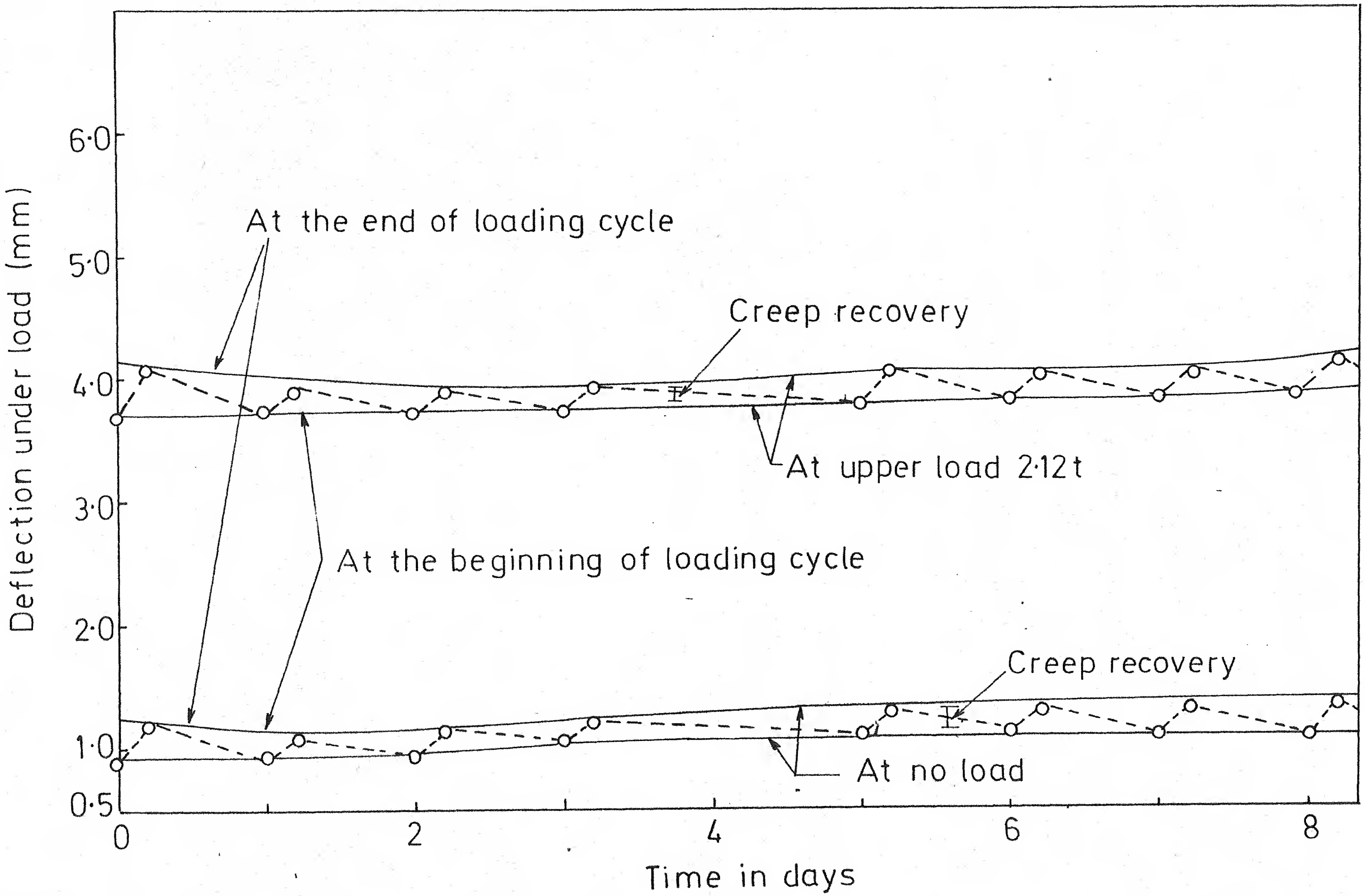


FIG. 3·6 CUMULATIVE DEFLECTION OF SPECIMEN B 25 (2ND)

increasing static load till failure and load deflection behaviour is shown in Fig. 3.7. The specimen failed due to failure of 25 Ø HSD bar and not through the weld.

3.4.2 VARIABLE PULSATING LOAD TEST :

Specimen C25 (companion beam of SC25) was tested under variable pulsating load and load-deflection behaviour during static loading and unloading (including peak load) is shown in Fig. 3.8. The specimen suddenly failed after 0.08 million cycles due to failure of 25 Ø HSD bar. The failure was just after the welding similar to that of specimen SC25.

Difference between the deflections under two loading points was not much and hence average of them was taken as the deflection under load.

3.5 STATIC AND PULSATING LOAD TEST RESULTS OF SPECIMENS SD25, D25, E25 AND E25 (2nd) :

Specimens SD25 and D25 were identical both having one ordinary 25 Ø HSD bar as tensile reinforcement. Similarly specimens E25 and E25 (2nd) were identical both having one butt welded 25 Ø HSD bar as tensile reinforcement. Butt welded joint was exactly at the middle of the specimen.

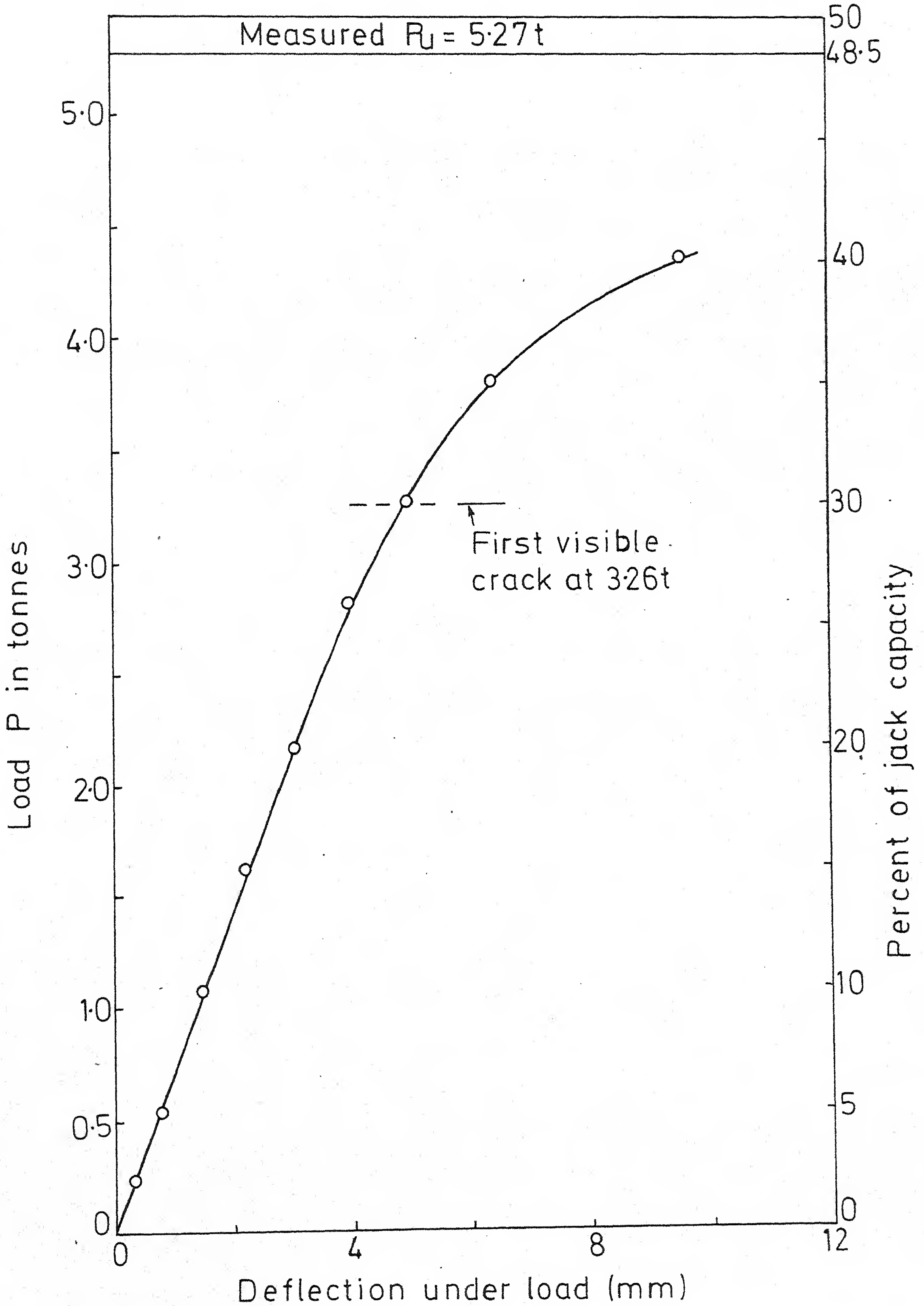


FIG.3.7 LOAD-DEFLECTION CURVE OF SPECIMEN SC 25

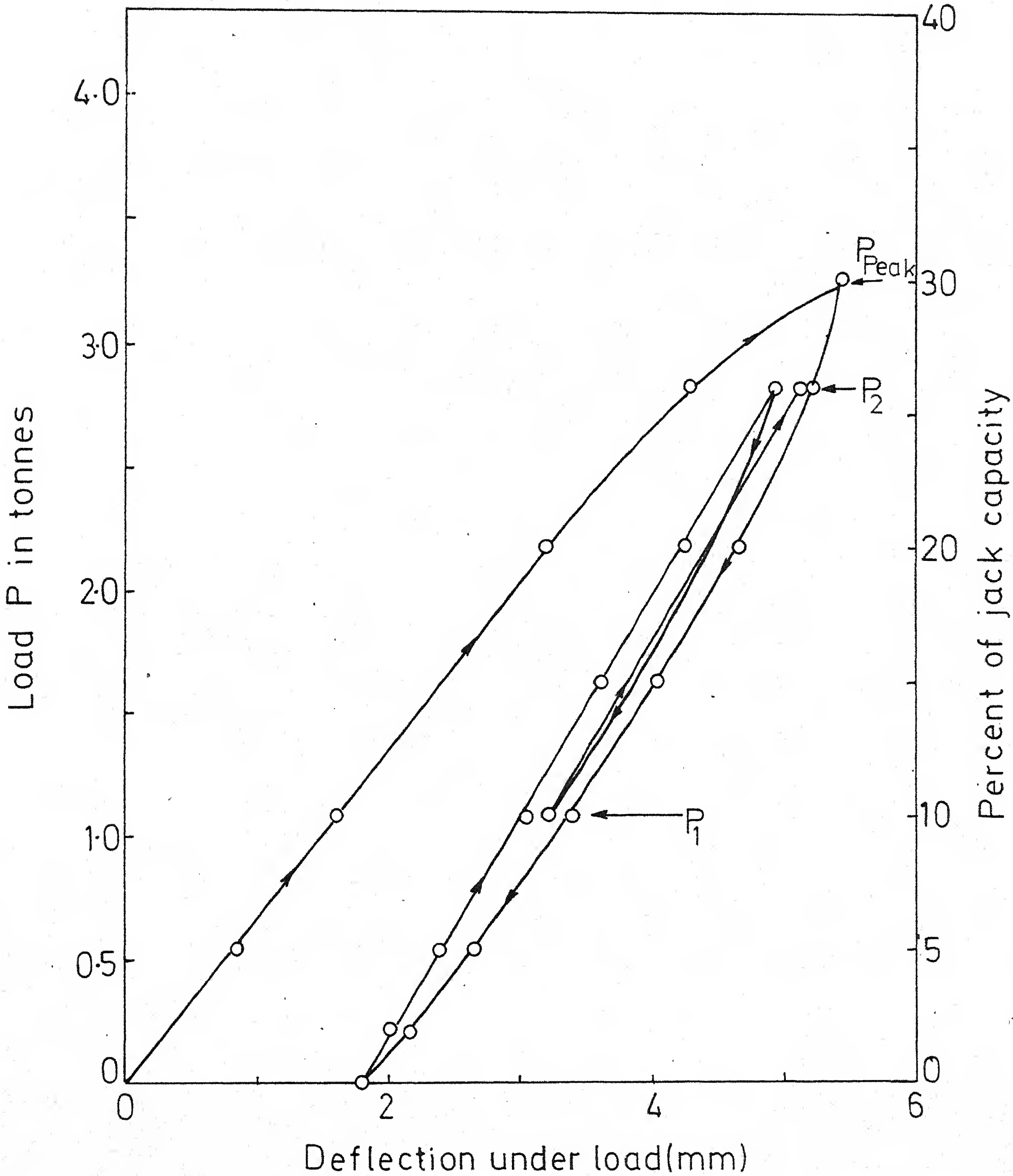


FIG.3·8 LOAD-DEFLECTION CURVE OF SPECIMEN C25

3.5.1 STATIC TEST :

Specimen SD25 was tested under monotonically increasing static load till failure. Load-deflection behaviour of the specimen is shown in Fig. 3.9. Ultimate failure of the specimen was due to failure of 25 Ø HSD bar.

3.5.2 VARIABLE PULSATING LOAD TEST :

Specimens D25, E25 and E25 (2nd) were tested under variable pulsating load and load-deflection behaviour during static loading and unloading (including peak load) of the three specimens are shown in Fig. 3.10, 3.12 and 3.13.

Specimen D25 was subjected to static loading and unloading corresponding to the following load levels :-

P_1 = 1.08 t i.e; 10% of Jack capacity

P_2 = 4.37 t i.e; 40.25% of Jack capacity

P_{peak} = 5.46 t i.e; 50.25% of Jack capacity

Severe cracks in the maximum bending moment zone were observed. Then the specimen was subjected to pulsating load ranging from P_1 = 1.08 t to P_2 = 4.37 t and rapid propagation of those cracks were observed within 25000 pulses. Then load levels were lowered to the following

limits :-

$P_1 = 1.08$ t i.e; 10% of Jack capacity

$P_2 = 3.91$ t i.e; 36% of Jack capacity

$P_{peak} = 4.89$ t i.e; 45% of Jack capacity

With these load levels the specimen was subjected to one million cycles of pulses and then tested under static load till failure for reserved strength. The post pulsating static test result and load-deflection curve is shown in Fig. 3.10. Ultimate failure was due to ultimate failure of 25 ∅ HSD bar (tensile reinforcement). Cumulative damage of the specimen is shown in Fig. 3.11.

Specimens E25 and E25 (2nd) failed after 0.5 million cycles and 0.2 million cycles respectively. In both the cases, tensile reinforcement(25 ∅ HSD bar) failed at the section next to the weld.

3.6 LOAD-DEFLECTION BEHAVIOUR :

Load-deflection behaviour of all the specimen was linear upto a certain load and became nonlinear afterwards. After the first visible crack the nonlinearity was prominent. It can be noticed from these figures that the slope of the later part of the first unloading curve is almost

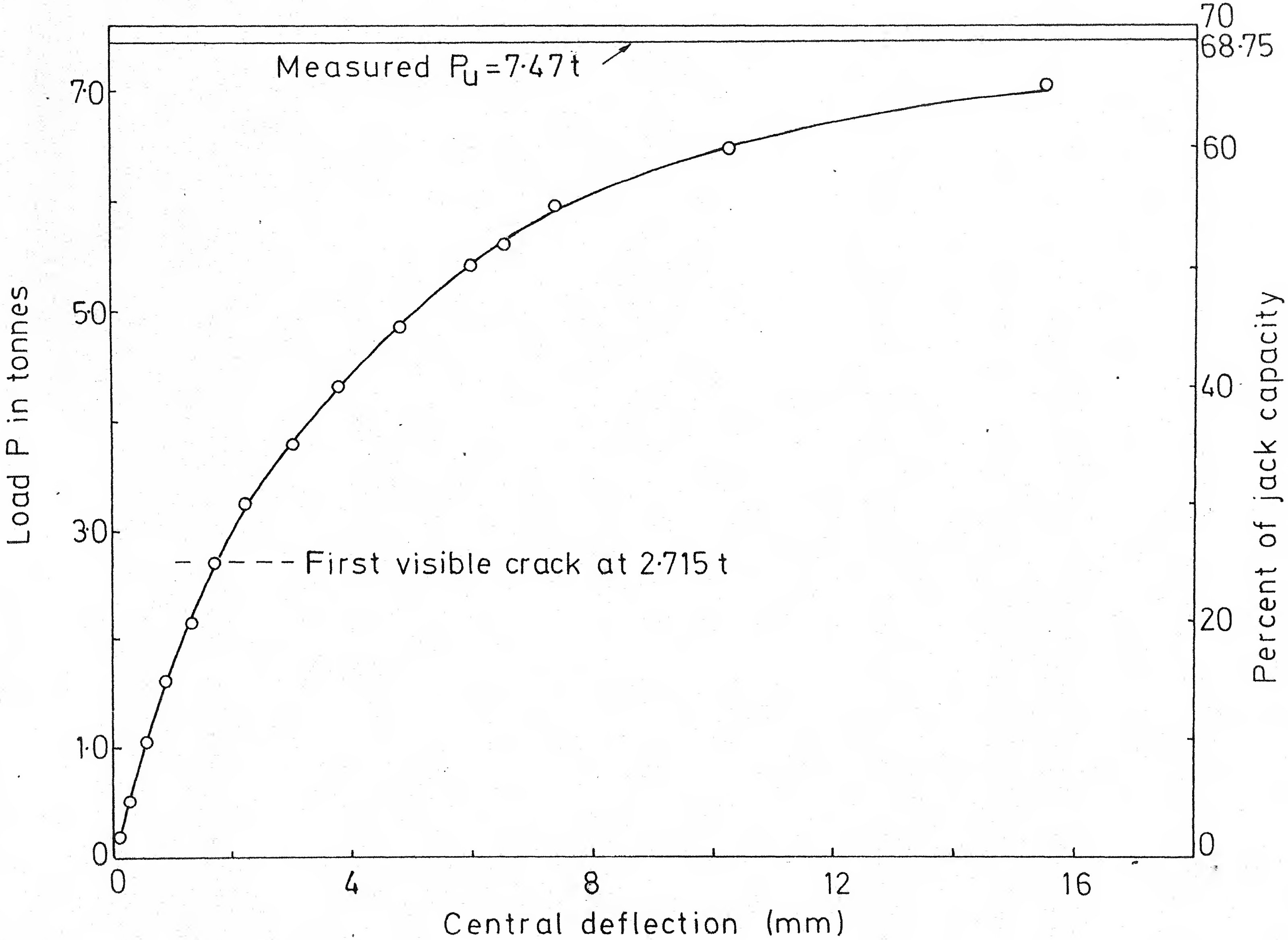
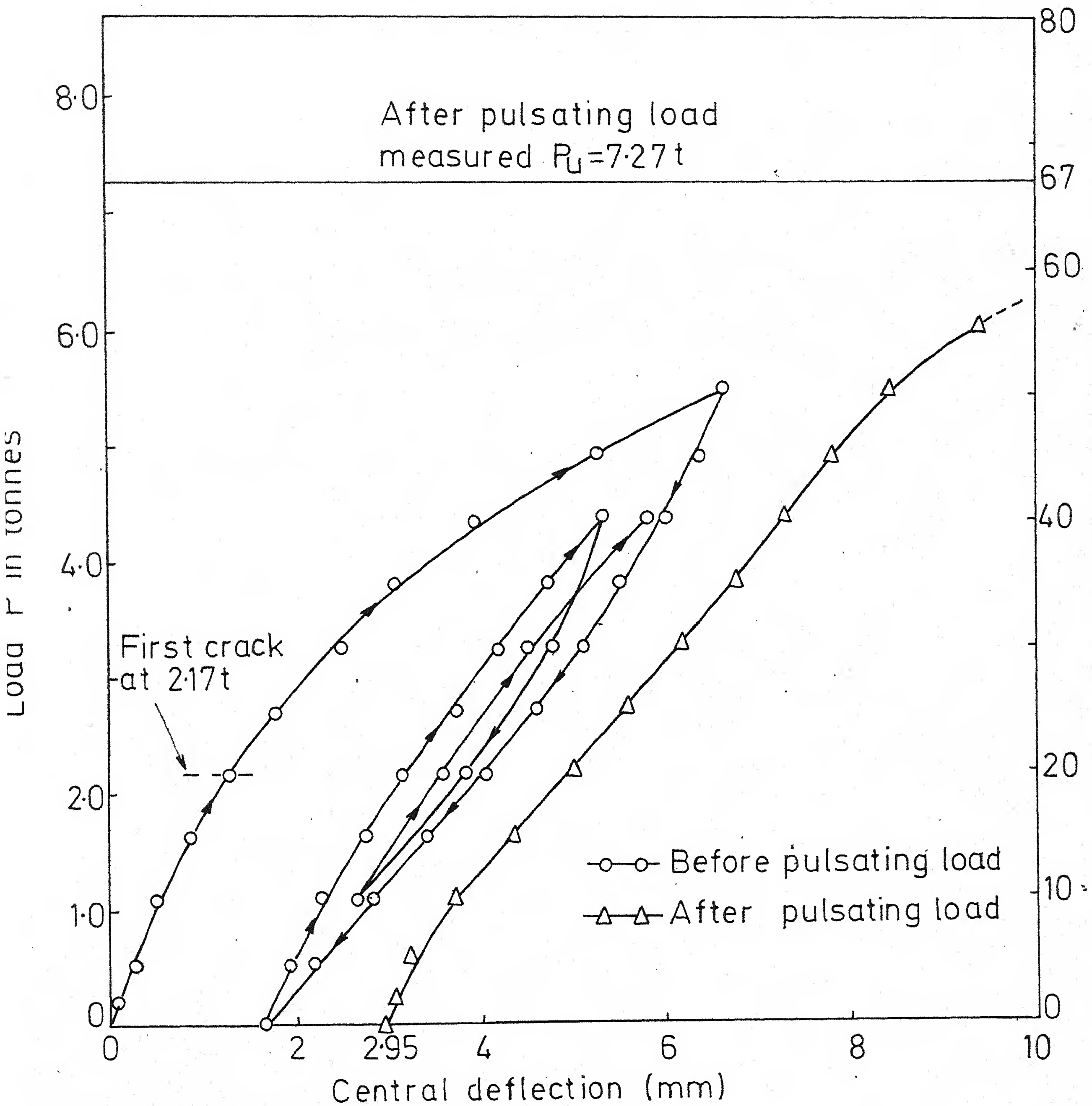


FIG. 3·9 LOAD-DEFLECTION CURVE OF SPECIMEN SD 25



FIG·3·10 LOAD-DEFLECTION CURVE OF SPECIMEN D 25

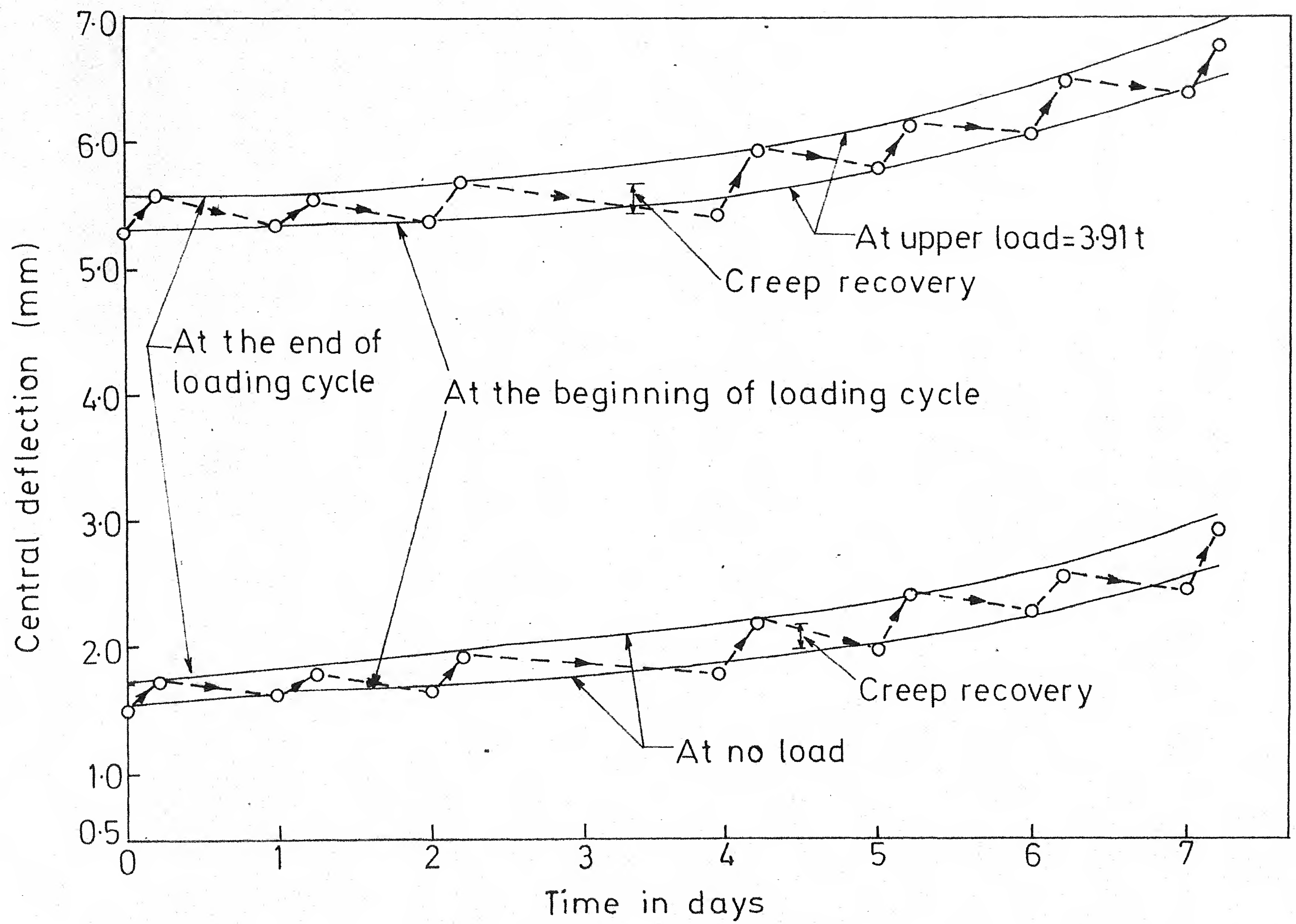


FIG. 3·11 CUMULATIVE DEFLECTION OF SPECIMEN D25

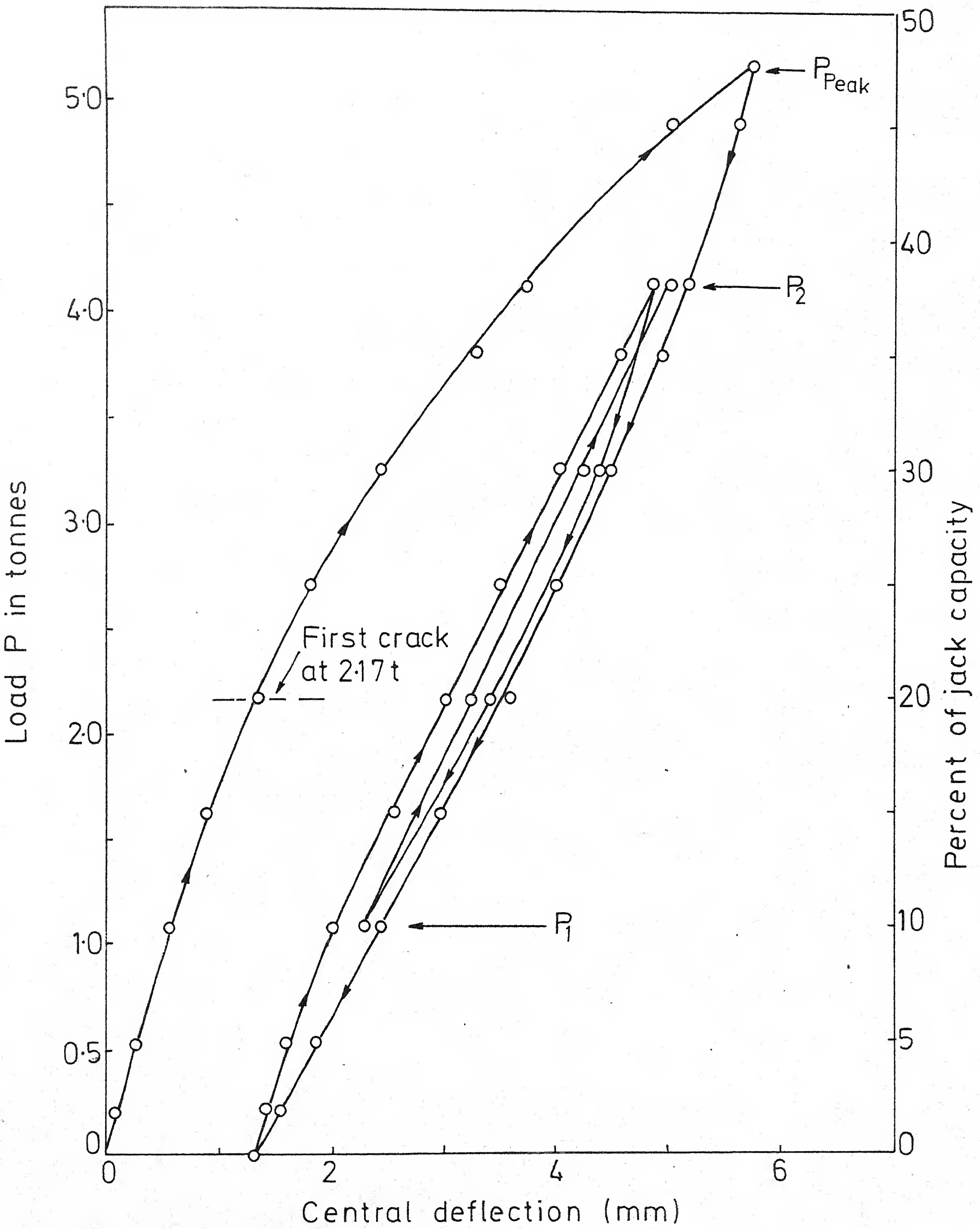


FIG. 3·12 LOAD-DEFLECTION CURVE OF SPECIMEN E25

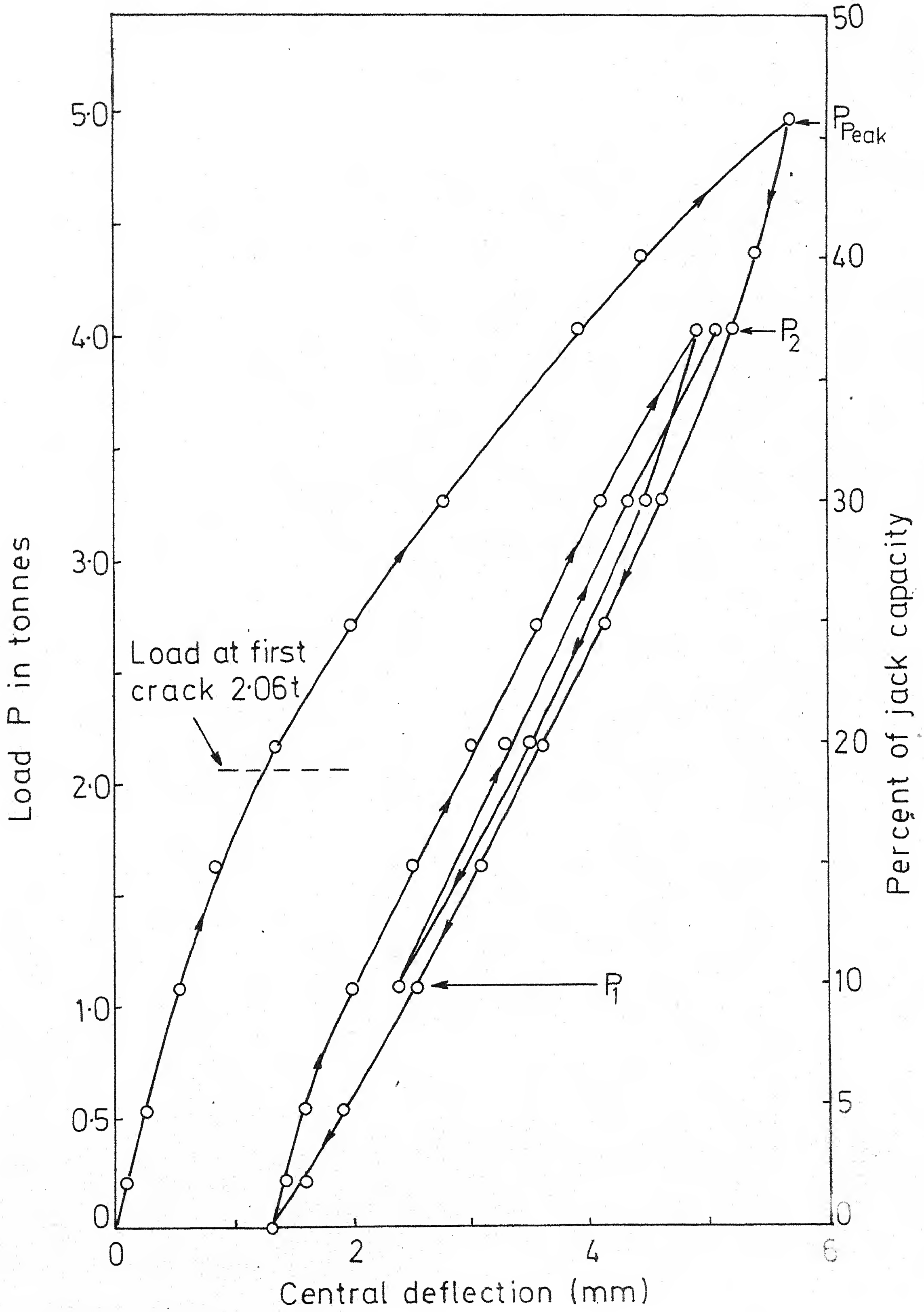


FIG. 3·13 LOAD DEFLECTION CURVE OF SPECIMEN E 25 (2ND)

same or is slightly less than the initial part of the first loading branch. Also the width of the loop formed by the load deflection curve becomes smaller with each cycle of loading at a given load level indicating stability of deflection.

3.7 CRACK PATTERN :

Crack pattern in all the hinged specimens was similar. Cracks for the specimen A25 is shown in Fig. 3.14. Those cracks may be due to the bond transfer for 25 Ø HSD bar to 10 Ø bars since 25 Ø HSD bar was just a little beyond the crack.

Cracks developed in all the rectangular beams were similar. Those cracks were at the middle of the beam. Some fine cracks were also found under the load points. Those cracks were vertical moment cracks. Cracks of specimens SD25 and E25 are shown in Fig. 3.16 and Fig. 3.17 respectively.

All the cracks of all the specimens widened during pulsating loads.

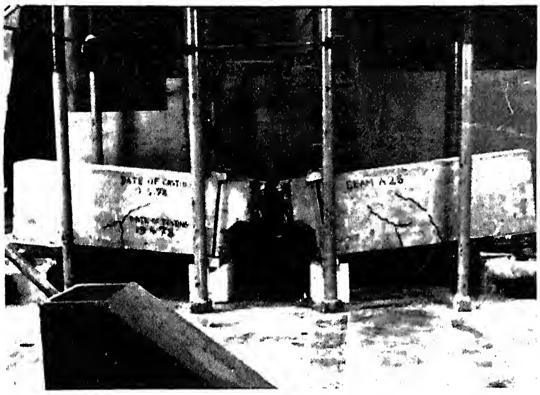


Fig. 3.14 Crack pattern in specimen A25

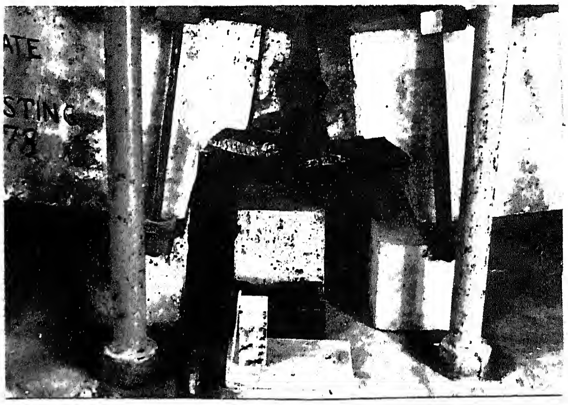


Fig. 3.15 Butt weld failure

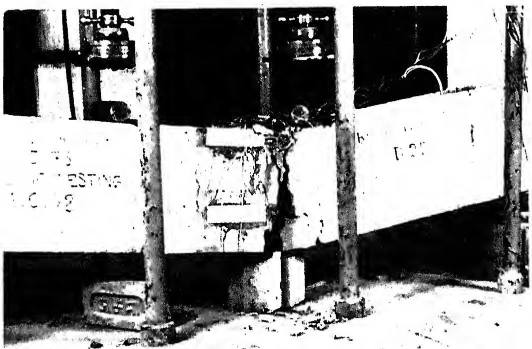


Fig. 3.16 Crack pattern in specimen SD25

Fig. 3.17 Cracks in specimen E25

3.8 CUMULATIVE DAMAGE AND ITS EFFECTS ON SERVICEABILITY :

Cumulative damage caused by the pulsating and overloads (peak loads) is represented by the relation between deflection and time as shown in Figs. 3.6 and 3.11. The deflections measured at the upper load level P_2 and no load level, before and after each pulsating load cycle, are plotted against time. The deflections at the end of load cycle are connected together (Figs. 3.6 and 3.11) indicating the cumulative damage due to the peak loads. Similarly the sustained deflection just before starting the load cycle are also shown in the figures. The ordinate between these two curves represent the amount of creep recovery.

From Fig. 3.6, it can be noticed that the damage was more or less constant throughout. The amount of creep recovery is more in the beginning, then reduces and then again increases very slowly with time. Also it can be seen that the cumulative deflection at the end of 8th loading cycle is about 1/1700th of the span length of the specimen, when it is subjected to the working stress level and peak stress level increased (Table 3.3) by 33%.

From Fig. 3.11, it can be noticed that the

cumulative deflection at the end of 7th loading cycle is about 1/760th of the span length when subjected to load levels 16.5 percent and 60 percent of ultimate load with overloads as high as 74 percent of ultimate load.

3.9 EFFECT OF SIMULATED LOADS ON STATIC ULTIMATE STRENGTH :

From Fig. 3.10, it can be noticed that after pulsating load upto 1 million cycle, specimen D25 was tested under static load. Measured ultimate load was 7.27 t against an anticipated load 6.555 t. This increase of ultimate load (P_u) is mainly due to the fact that ultimate strength of the bar is more than the yield strength on which theoritical results are calculated. It can also be due to strain hardening of 25 ∅ HSD bar.

3.10 STRESS LEVELS AND LOAD FACTORS :

Hinged specimens were subjected to pulsating loads. And corresponding to the load levels, stress levels developed are shown in Table 3.3.

25 ∅ HSD bar (without weld) subjected to stress levels 20%, 62% and 75% of ultimate stress (experimental) sustained 0.68 million cycles. But butt welded bar failed after 0.258 million cycles through weld when subjected to

19.8%, 57% and 76% of ultimate stress (experimental).

Butt welded bar, when subjected to the range of stresses 20%, 40% and 53% of ultimate stress (experimental) as lower, upper and peak levels respectively, sustained 2.43 million cycles, and the cumulative damage was very less.

Specimen D25 (with ordinary HSD bar) did not fail upto 1 million cycles when load levels were selected on the basis of load factors specified by IS 456 draft 1978. Theoritical ultimate load was calculated on the basis of characteristic proof stress 4250 kg/cm^2. Whereas specimens E25 and E25 (2nd) failed due to failure at the section next to the weld after 0.5 and 0.2 million cycles respectively. This happened probably due to the brittleness developed in the bar at the time of welding.

CHAPTER IV

CONCLUSIONS

4.1 INTRODUCTION :

Structures are subjected to live loads and over loads which are time dependent. Safety and serviceability of a structure subject to different types of loads is the primary aim of a structural design. Minimum safety with adequate serviceability should be the aim of structural design such that optimization of materials and overall cost of structure can be achieved. Moreover use of high yield strength deformed bars helps to reduce the area of reinforcements in R.C. structure which yields economy. Again in many cases, welding of reinforcing bars are done instead of providing bond or lap length to save material (particularly for higher diameter of bars) which yields again total economy of the structure. Hence, selection of minimum load factors or maximum permissible stresses that can be used in design of R.C. structures reinforced with high yield strength deformed bars with or without welding is needed.

In the present investigation an attempt is made towards the selection of load factors (also to check the

adequacy of load factors specified by different codes of practice) and permissible stresses for the safety and serviceability of R.C. structures reinforced with high yield strength deformed bars with and without weld under variable pulsating loads. Two types of weld joint (1) butt weld and (2) lap weld are considered. High yield strength bar of 25 mm diameter is chosen for the investigation since for higher diameter bars welding can save material.

Two types of beams (1) hinged beam (to develop direct tension) and (2) under reinforced rectangular beam (for testing under flexure) with butt weld, lap weld, and without weld, 25 ∅ high yield strength deformed bars were chosen. The effect of live loads and overloads were simulated by pulsator. The specimens were subjected to variable pulsating loads consisting of three limits (1) lower load (P_1) corresponding to the permanent load on the structure (2) upper load (P_2) corresponding to the live load and (3) Peak load (P_{peak}) corresponding to the overload (combination of dead load, live load and wind or earthquake load or any load over the normal working load). The pulsating load ranging from lower load (P_1) to upper load (P_2) with frequency 700 cycle per minute was applied continuously for 5 hours and peak load (P_{peak}) occured

twice at an interval of $2\frac{1}{2}$ hours. After five hours the specimens were left free from all external loads for 19 hours to minimize the fatigue effect.

Static tests were performed to know the ultimate capacity of the specimens. From the static test result, cube strength and deflection behaviour, the ultimate load capacity of specimen to be tested under pulsating load was anticipated by interpolation or extrapolation. Load ranges P_1, P_2, P_{peak} were selected from the load factors (specified by IS 456-1968 and IS456-draft 1978) and anticipated ultimate load (Appendix 'B').

4.2 CONCLUSIONS :

Direct tension test :

Proof and ultimate stresses of 25 Ø HSD bar (without weld) when tested under direct tension were found to be 4660 kg/cm^2 and 5778 kg/cm^2 respectively where as characteristic proof and ultimate stresses were 4250 kg/cm^2 and 4950 kg/cm^2. 25 Ø HSD bar (without weld) was tested under repeated tensile loading upto two cycles. Proof stress at 0.002 residual strain and ultimate stress were 4500 kg/cm^2 and 5500 kg/cm^2 respectively. Proof and

ultimate stresses were reduced marginally. Unloading and reloading paths formed a loop of small width. For ordinary mild steel bars unloading and reloading path are same upto the load termination point but not in the case of HSD bar.

Deflection :

For all the specimens deflection was linear upto first crack and became non linear afterwards. The width of the loop formed in between the unloading and reloading deflection curves decreased with each cycle of loading.

Crack pattern :

In all the specimens cracks were developed and propagated very slowly under pulsating loads. After some time crack propagation did not take place. Some fine cracks developed under pulsating load but they did not propagate.

Cumulative damage and effect on ultimate strength :

Cumulative deflection of rectangular beam reinforced with 25 Ø HSD (without weld) bar subjected to load levels 16.5%, 60% and 74% of P_u (anticipated) was of the order

of 1/750th of span length after 1 million cycle. Damage increased slowly as the number of cycles increased. After 1 million cycles the specimen was tested under static load till failure. Ultimate strength was found to be 7.27 t against an anticipated load 6.555 t. Increase in ultimate load is mainly due to the fact that ultimate strength of the bar is more than the yield strength on which theoretical ultimate load was calculated.

Cumulative deflection of the hinged beam with butt welded joint when subjected to stress levels 20%, 40% and 53% of σ^{E}_{ult} was 1/1700th of the span length after 2.43 million cycles. Damage increased slowly with number of cycles. At the beginning recovery was more, then reduced and again increased slowly and then remained more or less constant.

Stress levels :

25 ∅ HSD bar without weld, subjected to stress levels 20%, 62% and 75% of σ^{E}_{ult} (i.e. lower, upper and peak stress levels were 1181, 3602 and 4334 kg/cm^2 respectively) failed after 0.68 million cycles.

Butt welded joint with stress levels 1146, 3314 and

4400 kg/cm^2 failed through the weld after 0.258 m.cycles. This ensures that butt welded joint is weaker as compared to bar without weld. Butt welded joint subjected to stress levels 1163, 2292 and 3060 kg/cm^2 (i.e; 20%, 40% and 53% σ^E_{ult}) sustained 2.43 million cycles and cumulative deflection was very small. Permissible stress is 2100 kg/cm^2 for 25 Ø HSD bars. This permissible stress can be increased by 1.33 times i.e; 2793 kg/cm^2 for overstress of short duration. Hence butt welded joint, subjected to permissible working stress and overstress of 1.33 times permissible stress is safe when lower stress, level is around 1146 kg/cm^2. Specimen with lap welded joint failed suddenly after 0.08 million cycle.

Load factors :

Solid rectangular beam with unwelded HSD bar subjected to Load levels 16.5%, 60% and 74% of P_u (anticipated) sustained 1 m.cycles and the cumilative damage was 1/750th of span length. Load factors were chosen as per IS 456-draft 1978 and P_u (anticipated) was calculated on the basis of proof stress 4250 kg/cm^2. On the other hand when P_u (anticipated) was calculated on the basis of experimental proof stress 4660 kg/cm^2, severe cracks developed with the

load levels using the same load factors as per IS 456-draft 1978.

Other two beams reinforced with butt welded HSD bar failed due to failure at the section next to the weld. One of them failed after 0.5 million cycles with load levels 17.4%, 66.67% and 83.3% of P_u. Other one failed after 0.2 million cycle with load levels 17.9%, 66.67% and 83.3% of P_u. These two specimens failed due to the brittleness developed in bar at the time of welding.

The conclusions can be summarised as :

(1) Deflection is linear upto first crack and then becomes non linear.

(2) Cumulative deflection increases with the increase of number of cycles and magnitude of overloads.

(3) High pulsating loads of about 83% of ultimate load affected the ultimate load carrying capacity of the specimen.

(4) Butt weld and without weld bars are safe when subjected to permissible working stress and overstress 1.33 times working stress since cumulative deflection was very small and was of the order of 1/750th of the span length after

the end of 2.43 million cycle. The probability of failure by incremental collapse is very small when subjected to these stress levels.

(5) Load factors (as per IS 456 - draft 1978, i.e; 1.5 for D.L. and L.L. and 1.2 for combined loads) seems to be adequate when the ultimate load is calculated on the basis of proof stress 4250 kg/cm^2 and the beam is subjected to variable pulsating load.

(6) Butt welded HSD bars did not stand against the same load levels. The section of the bar next to the weld seems to be weaker since it failed under pulsating loads of 0.2 to 0.5 million cycles.

4.3 SCOPE OF FURTHER WORK :

There are many variable forces acting on the structures. The relation between the live load and dead load, live load and combined loads vary depending on several conditions. In case of small span bridges the ratio between dead load to live load could be 0.3 to 0.5. Whereas in the case of long span bridges, the same ratio is about 2 to 3. A fixed set of load factors for such variable levels of loads need extensive investigation. The capacity of a beam when subjected to pulsating loads

gets affected not only by the number of cycles but also by the range between the load limits. Therefore, investigation with lower load as a variable parameter is desirable. In the present investigation the lower load is kept in the range of 15 to 20 percent of that of the ultimate capacity. Selection of such a low level load as permanent load gives the safer bounds when compared to the higher values of this load.

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APPENDIX A

Hinged beams were chosen to develop direct tension on 25 Ø HSD bar. When hinged beams are subjected to two point loading as shown in Fig. 2.1, moment developed will produce compression on the hinge and tension in the bar.

Moment $M = P.l_1$

Resisting moment = T.S. (since from statics C = T) where

C = Compression developed on hinge

T = Tension developed in the bar

and S = Distance between centre of roller and the exposed bar (i.e; lever arm).

From equilibrium $M = P.l_1 = T.S.$

Hence $T = \dfrac{P.l_1}{S}$

Ultimate tensile load as per experiment = 28.02 tonnes.

To ensure the failure of the bar the following parameters were selected :

$l_1 = 70$ cm

$d = 22$ cm

$s = 14.5$ cm

and effective cover = 8 cm

Every details are shown in Fig. 2.1. For solid rectangular beams l_1 = 70 cm and d = 21.5 cm were chosen as shown in Fig. 2.2.

In all the specimens width was chosen as 20 cm. Design of reinforcements were done as per ultimate strength design specified by IS 456-1964. The specimens were designed not to fail in shear.

25 Ø HSD bar 1.5 meter long was provided in the hinged beam such that 600 mm bond length could be achieved.

APPENDIX B

Hinged beams : At each increment of loading plastic scale readings were recorded. Before starting the experiment distance 'S' (shown in Fig. 2.1) was measured. Distance 'S' varied with the loading and at any instant new distance S_1 was calculated from the scale readings. Tensile load and tensile stress was calculated by using the formulas

$$T = \frac{Pl_1}{S_1} \quad \text{and} \quad \sigma = \frac{Pl_1}{S_1 \ A_{st}}$$

where S_1 is the centre to centre distance between roller and HSD bar.

Stresses σ_1 , σ_2 and σ_{peak} corresponding to load levels P_1, P_2, P_{peak} were calculated as follows :

At the time of static loading before subjecting to pulsating loads the scale readings at each load increment was recorded. From those scale readings values of S_1 at different loads levels were calculated. $S_1^{(1)}$, $S_1^{(2)}$ and $S_1^{(3)}$ are the centre to centre distances between roller and HSD bar at load levels P_1, P_2 and P_{peak} respectively. Now stress can be calculated as

$$\sigma_1 = \frac{P_1 \; l_1}{S_1^{(1)} \; A_{st}} \; , \quad \sigma_2 = \frac{P_2 \; l_1}{S_1^{(2)} \; A_{st}} \quad \text{and}$$

$$\sigma_{peak} = \frac{P_{peak} \; l_1}{S_1^{(3)} \; A_{st}}$$

where l_1 = 70 cm and A_{st} = 4.85 cm^2

Specimen A25 :

Measured S = 13.2 cm

For specimen SA25, ultimate load developed was 4.89 t and measured S was 13.2 cm, P_{ult} for specimen A25 = 4.89 t is assumed.

Assuming C.L.F.= 1.5 for P_2 and C.L.F = 1.2 for P_{peak} and P_1 as 10% of jack capacity (since that is the minimum we can have in the pulsator for dynamic run) selected load levels are :

P_1 = 1.08t, P_2 = 3.27 t and P_{peak} = 3.91 t

$S_1^{(1)}$ = 13.18 cms $S_1^{(2)}$ = 13.10 cms $S_1^{(3)}$ = 13.02 cms

Therefore,

$$\sigma_1 = \frac{1080 \times 70}{13.18 \times 4.85} = 1181 \text{ kg/cm}^2$$

similarly σ_2 = 3602 kg/cm^2 and σ_{peak} = 4334 kg/cm^2

Calculations of all other hinged specimens are similar.

Solid Rectangular Beams : (as per Fig. 2.2)

Specimen D25 :

Data : Width (b) = 20.5 cm (average of width, measured at different sections)

σ_{cu} (at test date) = 360 kg/cm^2, σ_{yp}^{E} = 4660 kg/cm^2 (experimental)

$$M_U^T = 441.84 \text{ t cm.}$$

$$P_U^T = \frac{441.84}{70} = 6.31 \text{ t}$$

For specimen SD25 P_U^E = 7.47 t was found against P_U^T = 6.327 t

At 30% of jack capacity i.e; 3.258 t load, deflection of the specimen SD25 was compared with the deflection of the present specimen D25 at the same load and P_U of this beam was anticipated.

From Figs.3.9 and 3.10 at 3.258 t

deflection of SD25 = 2.2 mm

and deflection of D25 = 2.5 mm

Hence P_U anticipated $= 6.31 \times (\frac{7.47}{6.327}) \times (\frac{2.2}{2.5}) = 6.555$ t

From the load factors specified as per IS456 (draft 1978)

$$P_1 = 1.08 \text{ t}$$

$$P_2 = \frac{6.555}{1.5} = 4.37 \text{ t}$$

$$P_{peak} = \frac{6.555}{1.2} = 5.46 \text{ t}$$

Later on P_1, P_2 and P_{peak} was changed as discussed in Chapter III.

M_U^T was again calculated with $\sigma_{yp} = 4250$ kg/cm^2. From that ultimate load was anticipated.

$$M_U^T = 404.447 \text{ t. cm.}$$

and $P_U^T = \dfrac{404.447}{70} = 5.7$ t

$$P_U \text{ anticipated} = 5.7 \times \left(\frac{7.47}{6.327}\right) \times \left(\frac{2.2}{2.5}\right)$$

$$= 5.87 \text{ t}$$

Similarly,

$$P_1 = 1.08 \text{ t}$$

$$P_2 = \frac{5.87}{1.5} = 3.91 \text{ t}$$

$$P_{peak} = \frac{5.87}{1.2} = 4.89 \text{ t}$$

Calculations of other specimens are similar.